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Part III

BRIDGE DESIGN

BY

MANSFIELD MERRIMAN
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AND

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PREFACE.

THE present edition has been thoroughly revised and partly rewritten in order to bring the subject fully up to date, since the changes in bridge design and construction during the past ten years have been nearly as remarkable as during the preceding decade. The continued increase of live loads has led designers to give more attention to the stiffness of structures as well as to a more careful construction of details. The use of concrete piers has made it practicable to replace many pin-connected truss spans by shorter plate girder or riveted truss spans. Riveted trusses have been adopted so extensively and for much longer spans than formerly that in another decade but few pin bridges will be built for railroad traffic on main lines, except for very long spans over navigable rivers. The increasing use of the cantilever method of erection tends to eliminate eye-bars and pin connections still further, so that in a number of long spans now under construction most of the joints are riveted. The proportion of bridges with solid floors is steadily advancing, due chiefly to the effective use of plain and reinforced concrete to support the roadway.

Chapters IV, X, and XI (with the exception of Art. 126) and many articles have been entirely rewritten, and numerous changes were made in other parts of the text to bring it into accord with the latest practice. In Chapter IV the subject of erection is added, and in Chapter X the matter is all new, and largely represents recent developments in highway bridge construction in the Middle West. Plates I, II, IV, and VII are new and represent the revised standards of the respective railroads. The references to engineering literature in Chapters I,

VI, and VIII are considerably extended and brought up to date, while similar references are introduced in Chapters X and XI.

As stated in the preface to the first edition, the subject is presented "both rationally, as an application of the principles of mechanics, and practically, as an illustration of modern economic construction. Since probably more than ninety per cent of all bridges are those resting on two supports, this volume is confined to that class. For a beginner the study of bridge design should be largely that of proportioning details according to given specifications, and simple bridges furnish these in endless variety."

Grateful acknowledgments are due for kind assistance: to RALPH MODJESKI, H. E. STEVENS, and A. W. CARPENTER for permission to reproduce standard or general plans; to WILLIAM McNAB, F. H. BAINBRIDGE, JOSEPH O. OSGOOD, OLAF HOFF, J. E. GREINER, P. B. MOTLEY, and A. E. DEAL for photographs; to ENGINEERING NEWS and ENGINEERING RECORD for permission to reprint illustrations; to WARREN B. KEIM for the chapter on Fabrication and Erection; to F. O. DUFOUR for the chapter on Highway Bridges; to E. A. EVANS and P. W. PORTER for data on the analysis of weight for riveted truss bridges; and to many other railroad and bridge engineers for drawings which were examined to obtain the results given in Arts. 123, 124, 125, and 128.

A comparison with the fourth edition shows that the number of pages has been increased by 48, and the number of cuts by 36. In rewriting the volume, it has been the constant aim of the authors, not only to give the latest details of modern bridge practice, but also to set forth the reasons for such practice in a manner especially adapted to the needs of students and young engineers.

DECEMBER, 1911.

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CHAPTER I.

HISTORY AND LITERATURE.

ART. 1. EVOLUTION OF GIRDER BRIDGES.

All bridge structures may be divided into three classes, Beam Bridges, Suspension Bridges, and Arch Bridges. Beam bridges exert only vertical pressures upon the abutments or piers, suspension bridges exert a horizontal pull, and arch bridges exert a horizontal push in addition to the vertical pressures. Beam bridges include simple bridges, drawbridges, continuous bridges, and cantilever bridges. A simple bridge is one resting upon two supports; and probably over ninety percent of all bridges are of this kind. Parts I, II, and III of this work are devoted entirely to simple bridges, while the other forms are discussed in Part IV.

Simple bridges are of two classes, girder bridges and truss bridges. A truss bridge is one whose floor is supported by two or more frameworks, called trusses, each consisting of two chords, which are connected by bracing. A girder bridge, on the other hand, has its floor supported by solid or built-up beams. A wooden beam, a rolled I-beam, and a plate girder, formed by riveting angles and plates together, are examples of girders. Girder bridges are used for short spans, usually less

than 100 feet, while truss bridges are used for longer spans, and sometimes for spans as short as 50 feet.

Probably the first bridge was merely a tree-trunk that had fallen over a brook; later, several trees or logs placed side by side, and covered perhaps with brush and earth, formed a structure of greater convenience for the traffic of semi-civilized people. When the stream was too wide for a single span, a rude pier of piles or stones was built to support the ends of logs extending from it to the shore. Several piers of this kind were built for still wider streams, and thus arose the trestle structure, in which each span consisted of simple beams. The oldest wooden bridge on record, the famous "Pons sublicius," built across the Tiber, at Rome, about 650 B.C., is believed to have been of this kind, as also was the bridge built by CÆSAR over the Rhine in 55 B.C.

Little progress beyond the simple wooden beam was made until the early part of the nineteenth century, when cast-iron beams began to come into use. It was then soon recognized that the depth of the beam was a controlling factor in its strength, and that the greatest economy of material resulted by forming the cross-section so that the upper and lower parts should be thicker than the middle part. Thus arose the flanges and the web of a girder, the flanges carrying the greater part of the horizontal stress, while the web served mainly to hold the flanges together. Such cast-iron beams, with ⊥, ⊔, and [ ] sections, were used before 1840 for bridges on many English railroads, the longest span of a beam cast in one piece being 46 feet. These bridges, however, proved unsatisfactory on account of the low tensile strength and unreliability of the metal.

The first wrought-iron rolled beams were made in England about 1820 for railroad rails, and their use for other purposes slowly increased in both Europe and America, so that, by 1875,

I-beams up to 15 inches in depth were obtainable. Since 1890 medium steel has rapidly replaced wrought iron, so that now all I-beams are rolled of this material, and sizes up to 24 inches in depth and 30 feet in length are readily found in the market. Many deck bridges of 30 feet span or less have been built with such beams, and they also are extensively used for the floors of buildings and bridges.

About 1850 built-up plate girders, formed by riveting angles to a solid web plate, began to be used in Europe, and later were

Fig. 1.

introduced into this country, where they are now extensively employed for spans ranging from 30 to 100 feet. Fig. 1, from a photograph, shows the plate-girder bridge of 80 feet span, built in 1892 at Ithaca, N. Y., on the Delaware, Lackawanna and Western Railroad. The largest spans of plate-girder bridges are from 120 to 140 feet in length, the depth of the girders being about 12 feet. They are stiffer than truss bridges, and the shorter spans have advantages in erection, as a girder may

be made entire in the shop and swung into place by a derrick, the only field-riveting required being that necessary to connect the girders by lateral bracing.

A tubular bridge is a girder structure with its sides formed of plates and stiffeners, and its top of channels, angles, and plates, all being riveted together so as to form a closed tube. This type originated in England about 1840, and in 1850 STEPHENSON built the great Britannia bridge in Wales on this plan, the tube being 25½ feet high and 13¾ feet wide inside, and there being four spans, two of 230 feet and two of 460 feet. The Victoria bridge over the St. Lawrence River at Montreal, completed in 1859, was of this type, but it was replaced in 1898 by a truss structure. These tubular bridges, though stiff, were unnecessarily heavy, and accordingly very expensive, and the passage through them was like that through a tunnel. All experience indicates that the girder system of construction cannot be economically applied to bridges of long span.

A lattice truss, or lattice girder, as it is sometimes called, consists of flanges formed like that of the plate girder, but with the solid web replaced by flat, diagonal bars. The Warren truss, with a double system of web bracing (Part I, Art. 65), originated in England about 1840, and it may be regarded as being an attempt to economize material by removing unnecessary parts of the web. This was a step in the right direction, as the web stresses were thereby more closely determinate than before. But, as will be seen in the following articles, greater precision regarding stresses and greater economy in material have been attained by discarding the double set of diagonals, and using only a single system of bracing to connect the chords.

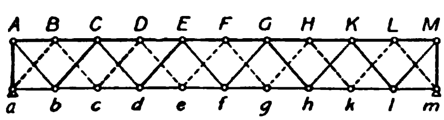

Fig. 2.

ART. 2. TRUSS DESIGN PRIOR TO 1800.

Bridge design prior to the year 1800, and indeed for many years after, was almost wholly empirical. Arch bridges of stone had been successfully built since the time of the Romans, and structures of timber were used for roofs and often for bridges, but the true idea of a bridge truss and of the functions of its members was not fully understood until near the middle of the nineteenth century. About 1830, owing to the introduction and development of railroads in both Europe and America, bridge construction assumed an importance never before known. In Europe the main line of evolution was based upon the metal girder, as described in the last Article. In America, however, the evolution was along the line of the truss, starting with timber and gradually developing into structures of iron and steel. A truss is a framework whose members are so arranged that they are subject only to longitudinal stresses of tension and compression. These members should be arranged in triangular figures so that no distortion of the structure can occur without bringing the proper stresses into action, and the applied loads should preferably be concentrated at the joints (Part I, Art. 18). The simple truss, supported at its two ends, is the one whose history is now to be considered.

The king-post truss shown at *a* in Fig. 3 may be supposed to be the origin of all modern bridge trusses. Prior to 1800,

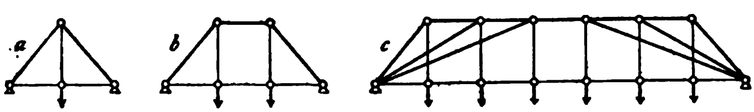

Fig. 3.

however, the principal line of development was that indicated by the diagrams *b* and *c*. On this plan many wooden bridges were erected during the seventeenth and eighteenth centuries. There were two chords, usually with a high camber, connected

by vertical timbers acting as ties to support the floor which was placed along the lower chord. From the top of each vertical an inclined brace was carried to the nearest abutment and the tops of the corresponding pairs connected by a straining beam. True truss action as we now understand it scarcely existed, the main idea being to carry each load to the abutment by the shortest route. This was a simple plan, but it proved uneconomical on account of the long braces whose stresses increase both with their length and the angle of inclination to the vertical. On this plan was built, in 1778, by GRUBENMANN, a timber bridge near Baden, which had the great span of 390 feet, and which exhibited much skill in carpentry.

The secret of economical and efficient truss arrangement lies in the panel system, which may be regarded as having been

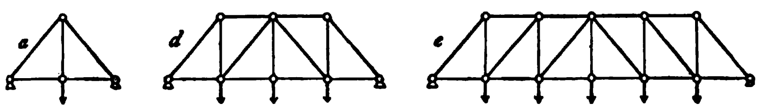

Fig. 4.

developed from the king-post truss in the manner shown in Fig. 4, where *d* is derived from *a* by the addition of a panel on each side, and *e* from *d* in like manner. This system was first used by PALLADIO, an Italian architect, about 1570, but it produced little or no influence on methods of construction, until it was rediscovered and used in the United States near the close of the eighteenth century by THEODORE BURR. The Burr truss may indeed be called the parent of nearly all the forms of bridge trusses now used in this country. Although so defective from the lack of counterbraces that it generally required the assistance of an arch to stiffen it under rolling loads, yet as it contained the sound principle of economy in a constant angle for the inclined members its panel system was transmitted to the Long truss, the Howe truss, and later to many other forms (Part I, Art. 63).

Concerning early timber bridges, as also for other valuable historical and descriptive matter, the student should consult COOPER's American Railroad Bridges, 1890, the article Bridge in the Encyclopædia Britannica, and the article Bridges in JOHNSON's Universal Cyclopædia, 1897.

ART. 3. PROGRESS FROM 1800 TO 1850.

Near the beginning of the nineteenth century many wooden bridges were erected in the eastern and middle states by THEODORE BURR and by TIMOTHY PALMER, both of whom used the panel system. PALMER's bridges generally combined the action of the truss and the arch in one structure, the lower chord being highly cambered, while BURR used the arch merely as auxiliary to the truss. The oldest truss bridge in the United States until 1909 was that built by BURR at Waterford, N. Y., in 1804, which was of hewn yellow pine, having four spans of 154, 161, 176, and 180 feet in the clear. Illustrations of this bridge and of one built by PALMER at Easton, Pa., in 1805 are given in COOPER's American Railroad Bridges. WERNWAG's great bridge of 340 feet span, built at Philadelphia in 1812, also deserves notice as a splendid example of early engineering work; the double diagonal bracing in its panels showing that probably its builder had considered the distorting action of rolling loads.

The lattice truss introduced by TOWN about 1820 consisted of planks pinned together, and was important only on account of ease of construction. In 1830, however, a radical advance was made in the true principles of truss arrangement through the introduction of panel counterbraces by S. H. LONG. In a pamphlet published by him in 1836 the function of counterbraces in preventing the distortion of the panels under rolling loads, and also their use in stiffening the truss when keyed up

so as to be under initial stress, is clearly recognized. Wooden Long trusses were erected on the Baltimore and Ohio Railroad as well as many for highway service.

In 1840 WILLIAM HOWE patented a combination truss having wooden chords and web diagonals and wrought-iron vertical ties, which has since been extensively used. Each panel had counter as well as main struts, both butting against cast-iron angle blocks. Many important bridges were built on this plan prior to 1850, the most notable being that over the Susquehanna

Fig. 5.

at Havre de Grace, Md., which had twelve spans of 250 feet each and a draw span of shorter length. The Howe truss is still in common use in localities where timber is cheap, and for short spans and light traffic it often makes an efficient and economical bridge. Fig. 5 shows a Howe truss bridge of several spans over the Stanislaus River, near Riverbank, Cal., and on the Atchison, Topeka, and Santa Fé Railway.

In 1844 the Pratt truss was introduced. In this a radical departure was made in the arrangement of the web members,

Fig. 6 Baltimore and Ohio Railroad Bridge crossing South Branch of Potomac River in West Virginia

the timber verticals being made to take compression, and the iron diagonals to take tension. This was a move in the direction of economy, since the length of the struts was decreased and thus the necessary cross-section somewhat diminished. Although at first built as a combination bridge, it never attained great popularity in this form, but soon after 1850 it began to be constructed entirely in iron, and in this form it has probably been more extensively used than any other form of truss. Fig. 6 shows a Pratt truss bridge of two spans, each 172½ feet long, erected in 1901.

Few iron structures were built in the United States prior to 1850, the first one being a span of 77 feet erected in 1840 over the Erie Canal, which was formed of cast-iron girders strengthened by wrought-iron rods. About the same time WHIPPLE built a truss with a curved upper chord of cast iron and a straight lower chord of wrought iron, forming the bowstring truss. A Howe truss in iron was introduced in 1844, and the Rider iron truss with a multiple web system was first built about 1847, but neither came into general use, and some that were built failed.

The first rational discussion of the determination of stresses and proportioning of cross-sections of truss members was published in 1847 at Utica, N. Y., by SQUIRE WHIPPLE under the title A Work on Bridge Building, in which are given methods of computing stresses due to dead and live loads, investigations as to the angle of economy for web bracing, with plans and details of the bowstring truss and of the double system Pratt truss, since known as the Whipple truss. WHIPPLE was far in advance of his time in rational views of economic truss design, but the circulation of his book was small, so that its influence was not fully exerted until several years after publication. He also built over twenty bridges on his plans which gave good service for many years. SQUIRE WHIPPLE is justly regarded

as the father of American rational bridge design. Drawings of bridges built between 1840 and 1850 may be seen in DUGGAN'S Stone, Iron, and Wood Bridges of United States Railroads, 1850; and also in HAUPT'S General Theory of Bridge Construction, 1851.

ART. 4. TRUSS EVOLUTION SINCE 1850.

The modern bridge truss is the result of an evolution or development in the sense that it exhibits those features which experience has found to be most economical and stable. Forms costly or unsafe have disappeared, while those cheap and strong have remained in use. Thus, the panel system has survived, while the method of transferring loads directly to the abutments by long braces, as seen in Fig. 3, has gone out of use. The Bollman truss, introduced soon after 1850, was an instance of the application of that erroneous principle, but it could not be built for spans greater than 160 feet, and even for shorter spans it was unable to compete in economy and stability with trusses of the panel system. The Fink truss (Part I, Art. 64) is another example of the use of that principle, and it too has disappeared.

The Whipple truss (Fig. 7) is an instructive instance of a form which was extensively used from 1850 to 1885, even for the longest spans, but which now is no longer built. This has

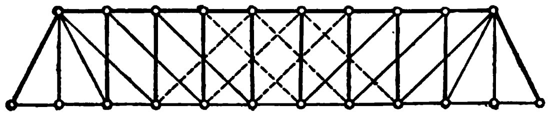

Fig. 7.

all the advantages of the Pratt type as regards the use of vertical compression members in the web, and also by the double system of webbing the panel points are brought nearer together, thus decreasing the length of the stringers, which for long

spans is a matter of importance. Stringers longer than 25 feet make an expensive floor; and this limits the economic depth of the Pratt truss to about 30 feet, and the span to about 300 feet, since it is not advisable to make the depth less than one tenth of the span. With the Whipple truss, however, using the same angle for the bracing, the depth of the truss can be doubled, and the span thus be economically increased. Many long bridges have been erected on this plan, among which may be mentioned the 515-feet span of the Ohio River bridge at Cincinnati, completed in 1877, and which at that date was the longest truss span ever erected. The Whipple truss began to go out of use merely because it was found to be more economical to support the floor beams by short sub-verticals attached to a single system of bracing than by the use of a double system, and because the single system is always more reliable and determinate in respect to stresses. The Post truss (Part I, Art. 66) is another example of a form once popular and now no longer in use.

The Warren or triangular truss was built with both single and double systems of webbing, but with a single system it afforded opportunity for the support of intermediate floor beams in a panel by the use of independent vertical members. In 1869 the channel span of 396 feet over the Ohio at Louisville was built on this plan, and in 1885 the 522-feet span at Henderson. This plan has been found advantageous because simplicity of truss action is secured, the only apparent disadvantage being the use of long inclined compression members in the webbing; in accordance with the law of evolution the former of these tends to be perpetuated and the latter to disappear.

At the present time the Pratt truss is most generally used for short spans. The Baltimore truss (Fig. 8) is used for both short and long spans; it possesses all the advantages of the Pratt, and in addition that of supporting intermediate floor beams by the use

of sub-verticals. The modified bowstring truss, shown in Fig. 9, uses the same idea, and here is gained the important advantage that the stresses in the chords are rendered closely uniform, as also those in the webbing. These elements combined

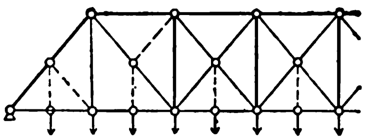

Fig. 8.

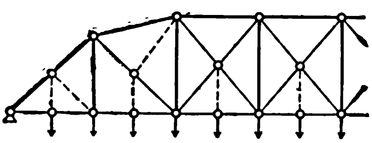

Fig. 9.

have rendered this form applicable to the longest simple trusses, the longest of all being in the great spans of 668 feet in the Municipal bridge (popularly called the Free bridge) over the Mississippi river at St. Louis, completed in 1911. Fig. 10 shows one span, 533 feet long, of the Delaware river bridge of the Pennsylvania Railroad, built in 1896.

To recapitulate, the principles which should control the arrangement of a simple truss are the following: first, the panel system whereby the inclined members in the webbing are kept

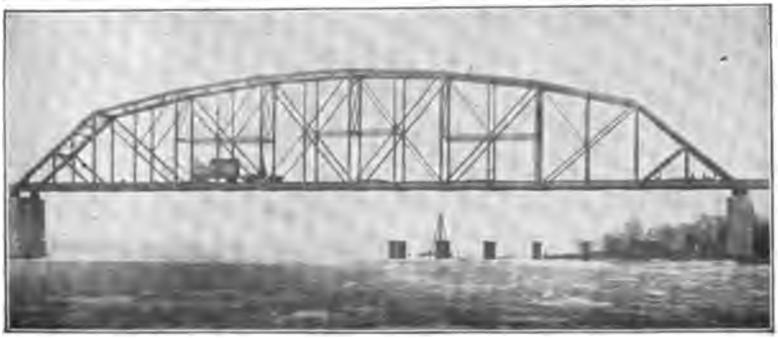

Fig. 10.

at approximately the same angle; second, the use of counterbraces to prevent distortion under a rolling load; third, that compression members should be made as short as possible; fourth, that a single system of webbing is always preferable,

and that intermediate floor beams may be supported when necessary by the use of independent verticals; and fifth, that the form of the truss should be such that the stresses in members of the same kind may be approximately equal.

In addition to the references at the close of Arts. 2 and 3 the following may be noted as treating of the development of trusses: Bridge Superstructure, a committee report in Transactions of American Society of Civil Engineers, 1878, Vol. 7, pp. 339–368; an Address by JOSEPH M. WILSON in Proceedings of the Engineers' Club of Philadelphia for 1889, Vol. 7, pp. 65–104; The Evolution of the Modern Bridge by CHARLES D. JAMESON in Popular Science Monthly, Feb. 1890, pp. 461–481; and the Evolution of Bridge Trusses by MANSFIELD MERRIMAN in Railway Age for May 19, 1893, Vol. 18, pp. 381–393.

ART. 5. MATERIALS USED IN BRIDGES.

Prior to 1840 wood was the material used in this country for bridge construction. Great skill in carpentry was developed to devise the joints, mortises, keys, and other connections, although little was known regarding the strength of the timber or the rational principles of designing the proportions of the parts. The Howe truss combined the use of wood and iron in a most simple and successful manner, wrought-iron adjustable tie rods being used for the vertical members of the web, while the wooden diagonals butted against cast-iron angle blocks. In the original Pratt truss, cast-iron joint connections were also employed, through which the wrought-iron diagonal ties passed. The first bridges wholly in iron had the compression members of cast iron and the tension members of wrought iron, this being, as WHIPPLE advocated, the best theoretic combination, since cast iron is high in compressive and low in tensile strength. Wrought iron, more-

over, was high in price, and could then scarcely be obtained except in the form of simple rods.

Bridges of cast and wrought iron were built extensively until about 1876, and many of short span since that year; but the numerous failures of the cast-iron parts led to the gradual substitution of wrought iron. Probably the first bridge in which both compression and tension members were made of wrought iron was that erected on the Lehigh Valley Railroad at Mauch Chunk in 1863, but in this cast-iron joint connections were used. Gradually but surely wrought iron displaced cast iron, both for truss members and for joint details, so that by 1875 cast iron was regarded as a material wholly inappropriate for use in bridge structures for railroad purposes, and the period of wrought-iron bridge development was at its height. But about this time steel began to be introduced.

The first extensive application of steel was in 1873 in the arches of the great St. Louis bridge, and later it was used in the trusses of the Brooklyn suspension bridge. In ordinary trusses it was at first employed in the form of eye-bars for tension members, and then for the webs of floor beams. But improvements in the methods of manufacture soon followed, so that by 1890 angles, beams, channels, and other shapes of medium or mild steel were easily obtainable in the market at the same price as those of wrought iron. This structural steel closely resembles wrought iron, but its strength is about ten or fifteen percent higher, and hence in 1900 it had entirely replaced wrought iron in bridge construction.

The average life of iron or steel railroad bridges is probably not far from twenty years, although under heavy traffic many are replaced after fewer years of service. They are liable to corrode from atmospheric influences and from the gases from the locomotives, while rivets and other connections are worn

and loosened under the frequently repeated stresses and shocks. Bridges built twenty years ago are now generally unable to carry the heavier rolling stock with a proper margin of security. Hence a metallic structure cannot compete with stone with respect to durability, and accordingly many roads are replacing short spans by arches of stone. The cheapness of iron and steel, however, generally renders metallic structures more economical in spite of their shorter life, and of course for long spans no other materials are available.

Some interesting notes by SQUIRE WHIPPLE on early iron bridges will be found in Railroad Gazette, April 19, 1889. A historical paper on steel manufacture in America by W. F. DURFEE is given in Popular Science Monthly, Oct. 1891, pp. 729–749. See also COOPER's American Railroad Bridges, originally published in Transactions American Society of Civil Engineers for 1889, Vol. 21, pp. 1–28.

ART. 6. JOINT CONNECTIONS.

The members of the early wooden bridges, such as the Burr truss and the Long truss, were connected together by means of joints devised especially for timber structures. The fish and scarf joints employed in the chords are still used in the Howe truss and in other wooden constructions, but most of the special devices of shoulders, mortises, and keys now exist only in a few isolated examples.

The combination trusses which next followed, like the Howe and Pratt, employed the method of screw connections to join the webbing to the chords. In the Howe truss the several pieces of the chords were bolted together laterally, and connected longitudinally by fish joints so as to form one continuous member, but the web struts butted against angle blocks and were held in place by screwing up the vertical iron tie rods.

The Pratt truss in its early forms had wooden chords upon which was placed at each panel point a cast-iron joint block, and through this passed the diagonal iron ties which terminated in screws and nuts by which the whole was held in place. This method was also extensively used in the Pratt trusses built of cast and wrought iron, and many special forms of screw connections were devised and employed. In general, however, most of these screw joints have gone out of use, on account of the greater cheapness and reliability of the methods of riveted and pin connections.

The riveted system of connections is the prevailing method of construction in Europe, but in this country it is mostly limited to plate girders and to lattice trusses less than 200 feet in span. In this system the chords are formed of angles, or channels, and plates, riveted together, with splice joints so as to make them practically continuous from end to end; and the web members are connected to the chords by rivets, either directly or by means of special plates riveted to both. The first riveted bridges in this country were erected on the New York Central Railroad about 1860, and the system has proved very serviceable there and elsewhere.

The pin system of connections is the one which has been most used and which has generally been regarded with the most favor by American engineers. At each panel point a pin, or round bar, passes through holes in the chord or web members and serves to transfer the longitudinal stresses from one member to another by means of the shearing and bending stresses generated in it. Some of the early bridges built by WHIPPLE had pins which passed through looped eyes in the tension members, but the first bridge which was pin-connected throughout was erected by J. W. MURPHY in 1859 on the Lehigh Valley Railroad at Phillipsburg, N. J. Wide forged eye-bars in connection with pins were first used in 1861 by

J. H. LINVILLE on the Pennsylvania Railroad. The system then rapidly spread on account of ease of erection, and thousands of pin-connected bridges are now in service.

Much might be said in comparison of the riveted and pin systems. Advocates of the former claim that it makes a stiffer structure and one less liable to accident from the failure of a single member. Advocates of the latter say that the stresses in the pin system are more determinate and that better workmanship is secured. But under present conditions the question of economy seems the controlling factor. A long span cannot be built as cheaply by the riveted system as by the other, and a short or medium span can sometimes be built more cheaply. Under proper specifications a good bridge can be designed and erected on either plan, and the item of cost will usually determine the decision. The riveted system generally requires a little more material than the pin system, and the latter requires more skilled workmanship. High prices for iron and labor were favorable to the development of the pin system, and as these become lower the riveted system comes more and more into use. The literature noted in the preceding articles contains much information regarding the various methods of joint connections. Further reference is made to the works named in the following pages, and also to a series of articles on Expired Bridge Patents by F. B. BROCK, in Engineering News during 1882 and 1883.

ART. 7. LITERATURE OF BRIDGE DESIGN.

The computation of stresses in the principal members of a bridge truss is the least part of the work of design, and hence books treating mainly on stresses are not noted in the following list. Bridge design includes of course the economic principles regarding the form of the truss, some of which have been mentioned in Art. 4, but more specifically it is the science of

details, that is, the proportioning of the members, the floor, the joints, and of all the splices, reënforcing plates, rivets, pins, and other parts which make up the structure. The list of books below includes such as treat wholly or in part of these topics, together with a few of historical and descriptive character. Although not complete, it is believed that it gives the works on Bridge Design most important for a college library and for the use of American students of bridge design. The list is arranged chronologically according to the date of the first editions.

POPE, T. A Treatise on Bridge Architecture. New York, 1811. This contains 196 pages of descriptions of early bridges, while the remainder is devoted to the author's "patent flying pendant lever bridge."

WHIPPLE, S. A Work on Bridge Building. Utica, N. Y., 1847, pp. 120 and 10 plates. The edition of 1869 contains also 128 pages of notes (printed by the author's own hands) explanatory of the original work. See Art. 2.

DUGGAN, G. Stone, Iron, and Wood Bridges of United States Railroads. New York, 1850. Consists mostly of drawings, with brief descriptive notes.

HAUPT, H. General Theory of Bridge Construction. New York, 1851, pp. 268 with 16 plates, giving examples of railroad bridges.

VOSE, G. L. Handbook of Railroad Construction. Boston, 1857, pp. 480. Contains 109 pages on wood, iron, and stone bridges.

HUMBER, W. Cast and Wrought Iron Bridge Construction. London, 1864, two volumes, with 80 plates, mostly descriptive of English bridges.

HEINZERLING, F. Die Brücken in Eisen. Leipzig, 1870, pp. 515. A historical and descriptive work on bridge develop-

ment in all countries. Also Die Brücken der Gegenwart. Leipzig, 1884, pp. 754 with 60 plates.

MERRILL, W. E. Iron Truss Bridges for Railroads. New York, 1870, pp. 130. A comparison of seven kinds of trusses with respect to theoretic economy.

BOLLER, A. P. Construction of Iron Highway Bridges. New York, 1876, pp. 144. Although written for the use of town committees, this book has been of much value to young engineering students.

DU BOIS, A. J. Strains in Framed Structures. New York, 1883, pp. 390 with 27 plates. This devotes 124 pages to design, and gives the complete design of a pin-connected bridge. The edition of 1896 has 209 pages on design and erection.

WADDELL, J. A. L. Designing of Ordinary Iron Highway Bridges. New York, 1884, pp. 244 and 7 plates. A book which has done much to improve the design of highway structures.

BENDER, C. Principles of Economy in the Design of Metallic Bridges. New York, 1885, pp. 195 with 9 plates. This does not treat of details, but gives critical theoretic comparisons of different forms of trusses.

RICKER, N. C. Construction of Trussed Roofs. New York, 1885, pp. 158. Mainly deals with stresses, but has two chapters on dimensions and details.

BURR, W. H. Stresses in Bridge and Roof Trusses. New York, 1886, pp. 454 with 12 plates. Devotes 112 pages to details and to the design of a railway bridge.

SCHÄFFER, T., and SONNE, E. Der Brückenbau (Vol. II of Handbuch der Ingenieur Wissenschaften). Leipzig, 1886–90, pp. 1812 with 77 plates.

HIROI, I. Plate Girder Construction. New York, 1888, pp. 94. Gives the design and estimate for a span of 50 feet.

MORANDIÈRE, R. Traité de la Construction des Ponts et Viaducs. Paris, 1888, pp. 1891, with 332 large plates.

COOPER, T. American Railroad Bridges. New York, 1890, pp. 58 with 27 plates. A historical and descriptive work of special value.

FOSTER, W. C. Treatise on Wooden Trestle Bridges. New York, 1891, pp. 160 with 38 plates. Gives many standard plans, accompanied by their bills of material.

JOHNSON, BRYAN and TURNEAURE. Modern Framed Structures. New York, 1893, pp. 517 with 37 plates. This gives 238 pages on details, with designs of several bridge structures.

WARREN, W. H. Engineering Construction in Iron, Steel, and Timber. New York, 1894, pp. 372 with 13 plates. Devotes 92 pages to the details and designs of simple span bridges, besides the designs of several other classes of bridges.

WRIGHT and WING. A Manual of Bridge Drafting. Stanford University, 1896, pp. 214 with 51 plates and 5 blue prints. Gives tables of shears and moments for girders, and details for different types of bridges.

WRIGHT, C. H. The Designing of Draw Spans. New York, 1897, pp. 93. This work relates mainly to the design of draw-bridge machinery. The 1898 edition has 320 pages.

BERG, W. G. American Railway Bridges and Buildings. Chicago, 1898, pp. 705. Gives many illustrations of details of timber structures, and other information compiled from reports of railroad superintendents.

WADDELL, J. A. L. De Pontibus: A Pocket-Book for Bridge Engineers. New York, 1898, pp. 403. Gives general specifications, and many tables and diagrams to facilitate computations.

HOWE, M. A. The Design of Simple Roof Trusses in Wood and Steel. New York, 1902, pp. 137 with 3 plates. Devotes about 50 pages to the design of roof trusses.

KETCHUM, M. S. The Design of Steel Mill Buildings and the Calculation of Stresses in Framed Structures. New York, 1903, pp. 494. Devotes 84 pages to the details and design of the framework of mill buildings and some miscellaneous structures, and 22 pages to specifications.

SKINNER, F. W. Types and Details of Bridge Construction. Part I. Arch Spans. New York, 1904, pp. 301. On the details of metal arch bridges. Part II. Plate Girders. 1906, pp. 424. Relates chiefly to the details of plate-girder bridges. Part III. Specifications and Standards for Short Railroad Spans. 1908, pp. 307. About 200 pages are devoted to standard designs and details.

THOMSON, W. C. Bridge and Structural Design. New York, 1905, pp. 93. Gives 46 pages to the stresses and design of bridge girders and trusses, and 17 pages to roof trusses.

KETCHUM, M. S. The Design of Highway Bridges and the Calculation of Stresses in Bridge Trusses. New York, 1908, pp. 565. Devotes about 190 pages to the design and details of steel bridge superstructures and about 80 pages to those of substructures and of masonry arches and culverts.

THOMSON, W. C. The Design of Typical Steel Railway Bridges. New York, 1908, pp. 185 with 5 plates. About 100 pages are given to the design of several bridges.

JACOBY, H. S. Structural Details or Elements of Design in Timber Framing. New York, 1909, pp. 377 with 2 plates. On the details and design of timber structures including roof trusses and some details of wooden bridges.

MORRIS, C. T. The Designing and Detailing of Simple Steel Structures. Columbus, 1909, pp. 209. About 170 pages are on the design and details of bridges, and 16 pages on roof trusses.

HUDSON, C. W. Notes on Plate-girder Design. New York, 1911, pp. 82 with 2 plates. Devoted almost exclusively to design.

SPOFFORD, C. M. The Theory of Structures. New York, 1911, pp. 373. Gives 102 pages to the design of beams, plate girders, and bridge trusses.

A number of monographs on large bridges have also been issued in book form, which are of special value to advanced students and engineers. Among these are The Quincy Bridge, by T. C. CLARKE, 1869; The Kansas City Bridge, by O. CHANUTE, 1870; A History of the St. Louis Bridge, by C. M. WOODWARD, 1881; The Washington Bridge, by W. R. HUTTON, 1889; The New Omaha Bridge, The Cairo Bridge, The Bellefontaine Bridge, The Memphis Bridge, and Others, by G. S. MORISON, 1889–94; The Thames River Bridge, by A. P. BOLLER, 1891; The Thebes Bridge, by ALFRED NOBLE and RALPH MODJESKI, 1907; The Cambridge Bridge Commission Report, 1909; and The Vancouver-Portland Bridges, by RALPH MODJESKI, 1910.

The Transactions of the American Society of Civil Engineers contain many papers both descriptive and critical. Of the latter class may be noted 'Specifications for the Strength of Iron Bridges,' by JOSEPH M. WILSON, in 1886, vol. 15, pp. 410–490; 'Some Disputed Points in Railway Bridge Designing,' by J. A. L. WADDELL, in 1892, vol. 26, pp. 77–282; and 'The Launhardt Formula and Railroad Bridge Specifications,' by H. B. SEAMAN, in 1899, vol. 41, pp. 140–268. Similar papers may be found in the transactions of various local engineering societies.

The Proceedings of the American Railway Engineering Association contain many valuable reports relating to specifications, details, and design for steel and wooden bridges. These include the annual reports of the standing committees on Wooden Bridges and Trestles and on Iron and Steel Structures, and the subsequent discussions.

The volumes of Engineering News, Railway Age Gazette, Engineering Record, and other technical periodicals contain

numerous articles, both theoretical and descriptive, on bridge design, and some of these will be mentioned in the following chapters. The Index of Engineering Literature, published by the Association of Engineering Societies, in 1892, and by the Engineering Magazine, in 1896, 1902, etc., gives many pages of titles of such articles, with brief notes of their contents; and this should be at the hand of every student who desires to become well informed on the progress of bridge development. But it cannot be too strongly urged upon the student to form the habit of making his own catalogue of articles, and of giving under each title his own synopsis of its contents and conclusions. By so doing he acquires a training in technical literary work which will be of the greatest value in promoting his professional advancement.

CHAPTER II.

PRINCIPLES OF ECONOMIC DESIGN.

ART. 8. DATA OF THE DESIGN.

In order that the most economic design may be made for a bridge it is necessary that complete data regarding its location should be known. An accurate map of the locality, showing the neighboring roads or streets, should be prepared, as also a profile of the crossing, giving the high and low water marks of the stream and the character of the earth or rock below its bed. This profile should be extended some distance from each bank of the stream in order to enable the approaches of the bridges to be properly arranged. The location of the bridge and of its abutments and piers are to be shown on the map, while the grade line of the bridge and its approaches are given on the profile. If there are more spans than one, the position of the piers is determined by making approximate estimates of their cost in different positions and then applying the principles of Art. 9.

In locating the abutments and piers it is always advisable to avoid a skew, as thereby the cost of the superstructure will be increased. When this cannot be done, as in the case of one street crossing another obliquely or in the case of a stream with rapid current, the angle of skew should be made as small as possible and the same in amount at each end of a span. In locating the grade line of the floor of the bridge the clear waterway desired is to be considered, as also the grades of the approaches; these will also determine whether the bridge is

to be a through or a deck structure or whether certain spans should be through and others deck.

Facts regarding the regimen of the stream, such as its velocity at both low and high water, its liability to freshets at different seasons of the year, and the amount of drift carried during freshets, are useful to a bridge company in estimating the cost of erection. The distance from the bridge site to the nearest railroad siding should also be stated in order that estimates of the cost of cartage may be made. The loads to be carried by the bridge, the lateral clearance required between trusses, and the vertical clearance needed for through bridges must be carefully specified. The kind of floor desired, the width and number of sidewalks, if any, should be stated. With these facts on hand the engineer is ready to prepare a general plan for both substructure and superstructure and to write specifications from which the detailed designs may be prepared. Time spent in gathering data is always usefully employed, for experience has shown that most of the mistakes and losses that have occurred in bridge construction have been due to imperfect knowledge of the local conditions.

ART. 9. NUMBER OF PIERS AND SPANS.

When a bridge is to be built across a river, one of the first considerations is that regarding the number of spans. This question is to be decided by the principle that the total cost of the substructure and superstructure shall be a minimum. In any event there will be two land abutments; and if the distance between these be short, no intermediate piers are advisable. Yet it is seen even here that if piers could be erected without any expense, it would be best to use them. Thus the relative cost of piers and their connecting spans determines the number of piers and spans which can be most economically built between the two abutments.

An old rule for this case states that the cost of the superstructure must equal the cost of the substructure in order that the cost of the whole may be a minimum. The cost of piers is to be determined by careful surveys and estimates for various locations along the line, while the cost of spans of different length may be approximately ascertained by consulting builders. A comparison of the different possible arrangements determines the most economic plan which sometimes agrees well with this rule.

The cost of common bridges is closely proportional to their weights. If l be the length of span, the formula $W = al + bl^2$ gives a good approximation to the weight (Part I, Art. 20), a and b being constants for the same type of truss. In this, al represents the weight of the track and floor system, while bl^2 represents the weight of the main trusses and lateral bracing. For example, the total weight of steel in pounds in a single-track railroad riveted bridge (not including cross-ties and rails) varies from $700\,l + 7\,l^2$ to $1000\,l + 10\,l^2$, where l represents the span of the bridge in feet.

If the cost of piers is about equal, and they be spaced at equal distances apart, the following investigation will give the economic number of spans. Let L be the total distance between end abutments, x the number of spans, and hence $x - 1$ the number of piers, m the cost of the two abutments, n the cost of each pier, and p the cost per pound of the bridge superstructure. The weight of the x spans, each of length $\frac{L}{x}$, is then $x\left(a\frac{L}{x} + b\frac{L^2}{x^2}\right)$, and the total cost of the work is

$$C = m + n(x - 1) + p\left(aL + \frac{bL^2}{x}\right).$$

This will be a minimum when the first derivative of C with respect to x becomes zero, and this gives $n = pb\frac{L^2}{x^2}$, which shows that the cost of one of the intermediate piers should

equal the cost of the main and lateral trusses of one of the spans. Or, $x = \sqrt{\frac{pbL^2}{n}}$ gives the economic number of spans. For example, if $L = 1000$ feet, $a = 900$, and $b = 9$ for pin-connected spans, and $p = 4$ cents per pound, then for $n =$ \$8 000, the most economic number of spans is $x = 7$, and the total cost is \$135 400, exclusive of abutments. Here the cost of the piers is \$48 000, and that of the seven spans is \$87 400, which indicates that the old rule may sometimes be at fault. Again, if the cost of a pier be $n =$ \$12 000, the economic number of spans is $x = 5$, which gives \$48 000 for the piers, and \$108 000 for the superstructure.

When the cost of piers varies in different parts of the river, the spans will vary in length, the shortest ones generally being nearest the banks. For each possible case a rough estimate of the cost of piers and spans may be made, and thus the arrangement which gives the minimum cost may be determined. For example, suppose the distance between abutments to be 500 feet, a pier near the middle costing \$6000, and piers within 150 feet of the shore costing \$4000 each; then, using the above values of a, b, and p, the cost of one pier and two 250-feet spans would be \$69 000, while the cost of two piers, with a middle span of 200 feet, and two side spans of 150 feet, would be \$56 600.

ART. 10. CHOICE OF KIND OF BRIDGE.

Whether the bridge span is to be deck or through will be determined in each case by the local conditions, among which the grades of the approaches are controlling factors. A deck span is usually cheaper than a through one, since the width of the bridge may be less and something is also saved on abutments and piers, and should hence be chosen if the approaches allow it and proper waterway can be secured beneath it.

The width of the bridge between trusses is determined by the amount of traffic. For a single-track railroad this width for a through bridge is taken as 14 or 15 feet in the clear, while for a deck bridge 10 or 12 feet between centers of trusses is usually enough for short or medium spans.

The cost of the bridge is a material factor in determining the kind which is to be erected, and the problem of selection is hence a very complicated one. For railroads experience has led to the conclusion that at present the best results both as to stability and economy are obtained by using solid rolled beams for short spans up to 20 or 30 feet, plate girders for spans from 25 to 110 feet, riveted lattice trusses for spans from 100 to 200 feet, and pin-connected trusses for spans over 160 feet. It will be observed that these figures overlap each other, indicating that there is no distinct line of demarcation between the length of spans of the different classes, and detailed designs and estimates are often required to determine the cheapest type.

The particular kind of truss is not usually stated in the specifications, this being left to the bidders who often may present plans which differ materially in general appearance. If all these plans conform to the specifications, the contract is awarded to the lowest responsible bidder. The choice of the kind of truss is hence usually made by the sellers rather than by the buyers of bridges, but the question of accepting the tender of the lowest bidder is sometimes influenced by the form of truss adopted in his plan.

The discussion in Art. 4 gives only the general economic conditions which determine the form of truss. The depth of the truss is to be selected so as not only to secure proper headway and afford opportunity for cross-bracing, but also so as to give the least amount of material; this question of economic depth is investigated in Art. 11. The number of panels should be odd rather than even for best economy, and should be such

that the panel lengths, or distances between floor beams, may range from 12 to 24 feet. Probably the best panel length, as far as the floor system is concerned, is that which renders the weight of a floor beam about equal to that of the stringers in one panel.

Æsthetic considerations should not be overlooked in choosing the kind of bridge, and the old maxim that strength, beauty, and economy go together contains some truth. The parabola is a line of beauty, and through trusses having the upper chords broken or curved are among those which now seem to possess the highest degree of economy for spans between 100 and 550 feet. In deck trusses, however, the upper chord is necessarily straight, and the slight downward curvature sometimes given to the lower chord does not appeal to the public as an element of beauty. For deck bridges arches are always more beautiful than trusses, but unfortunately their cost is much greater.

Approximate economic comparisons of trusses of different forms may be made by comparing the theoretic amounts of material, the material in any member being taken as proportional to the product of its maximum stress by its length. Investigations of this kind were first made by WHIPPLE in 1847, and have since proved of value in studying the question of economic proportions. But such investigations are of limited value in comparing the relative economy of different forms, unless the unit stresses for compression be taken less than those for tension, and as required by a formula for columns. To introduce this element in a theoretic comparison leads to great complexity, and it is, in fact, only by making actual designs from a given specification that reliable results can be obtained. The work of BENDER, cited in Art. 7, and CREHORE's Mechanics of Girder (New York, 1886) may be consulted for examples of such investigations.

ART. 11. ECONOMIC DEPTH.

The economic depth of a girder or truss is that which renders its weight a minimum. Such a depth exists by virtue of the facts that the chord material decreases and the web material increases as the depth is increased. For a plate girder it is a rough general rule that the economic depth obtains when the weight of the flanges is equal to the weight of the web. To show this it must be borne in mind that the thickness of the web plate is practically constant for a girder of short span, being rarely greater than $\frac{5}{8}$ nor less than $\frac{3}{8}$ inch. The material in the web hence varies as $a \times h$, and that in the flanges as b/h, where a and b are constants depending on the span loads and working unit stresses. The total material may then be represented by $a \times h + b/h$, which is a minimum when the two terms are equal, that is, when the flange weight equals the web weight, or, more strictly, when the cost of the flanges equals the cost of the web. In practice, however, the weight of the flanges often exceeds that of the web. (See Arts. 66 and 71.)

For a truss an approximate determination of economic depth may be made by computing the stresses in terms of the panel length and depth, multiplying each stress by the length of the corresponding member, and regarding the products as representing the amounts of material, and then finding the depth that renders the sum of these products a minimum. For example, take the PRATT truss of which one-half is shown in Fig. 11. Let the dead load per panel point be

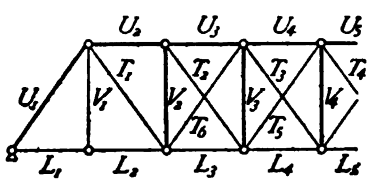

Fig. 11.

w, and the live load $3\,w$. Let the panel length be p, and the depth of the truss be h. By the methods of Part I the maxi-

mum stress in each member due to the given load is computed, and each stress is then multiplied by the length of the corresponding member. For example,

MEMBER.	STRESS.	STRESS × LENGTH.
L_1	$16\,wp/h$	$16\,wp^2/h$
U_1	$16\,w(h^2+p^2)^{\frac{1}{2}}/h$	$16\,w(h^2+p^2)/h$
V_1	$4\,w$	$4\,wh$
....	....	....

and the sum of all the products in the last column will be found to be

$$\text{Sum} = \left(70\,h + 287\tfrac{2}{3}\,\frac{p^2}{h}\right)w,$$

which represents the amount of material in one-half the truss. Differentiating this expression with respect to h and equating the derivative to zero gives $h = 2.03\,p$, or the theoretic economic depth is about twice the panel length. For this depth the web material, including the end posts, is about 165 wp, and the chord material about 118 wp, the former being about 40 percent greater than the latter. This value of the economic depth is, however, considerably too large for practice, since the investigation has neglected the increase of the amount of material necessary in compression members.

It may be further noted that great exactness in regard to economic depth is not important, since a function changes slowly in the vicinity of a maximum or minimum, so that considerable variations in depth may be made without much increasing the quantity of material. For instance, in the above case the following shows how the material varies for different depths:

Depth h =	1.8	1.9	2.0	2.1 p,
Material =	285.3	284.4	283.8	284.0 wp,

which indicates that the depth may vary ten percent from the economic depth without increasing the material as much as one percent.

Lastly, it may be noted that there has been a constant tendency since about 1875 to build through truss bridges with greater and greater depths. This has resulted from considerations of stiffness as well as those of economic depth. Increasing the depth of a truss diminishes its deflection under live loads and thus decreases the injurious oscillations which wear out a railroad bridge. This tendency is apparent in both long and short spans, but especially in the shorter ones. In some cases the increase in depth has gone so far as to require the vertical posts to be stiffened by horizontal braces placed between them in the plane of the truss and midway between the upper and lower chords.

ART. 12. PRACTICAL CONSIDERATIONS.

The engineer who draws the specifications is primarily responsible both for the strength and security as well as for the economy of the structure. For, if improper working stresses are prescribed, or proper rules for stability are omitted, the builders, under the influence of competition, will present plans of structures lacking in security; or, if excessive and unusual requirements are made in the specifications, the plans presented will not be economical. At present there are so many specifications which may be called standard that it is not possible to go far astray in either of these directions, particularly for railroad bridges. For many highway structures, however, the specifications are very loosely drawn, and every year there are erected some bridges which are defective either in stability or economy. As a general rule economy demands a bridge of proper stability, and the proper degree of stability will be secured by structures of the best economic design.

The designer should, of course, strictly follow the specifications, yet in details and dimensions he has great liberty of choice. He should be well acquainted with the market sizes of materials and with the market prices. Variation from regular sizes always involves delay and extra cost. Uniformity of sizes is advantageous, since several things of one kind can be purchased or made more cheaply than if they are of different dimensions. Simplicity of connections should be studied not only with respect to strength, but also with regard to economy of manufacture. The lines of action of all stresses meeting at a joint should intersect at a point, in order to avoid secondary stresses of twisting or bending. Simplicity, as a rule, leads to both determinate stresses and the economy of material.

In riveted work excessive nicety in the spacing of rivets should be avoided. If possible the pitch should be in even inches, that is either 2, 3, 4, 5, or 6 inches, especially when the rows are long, as in columns and the flanges of plate girders. It will be more economical still if the pitches can be reduced to two, 3 inches and 6 inches, but this is not so easy to attain and still maintain the proper uniform strength throughout.

In pin-connected work it will often be advantageous, particularly for short spans, if the pins are of uniform sizes, except perhaps those at the ends. As the strength of a pin depends more upon its resistance to transverse stresses than to shearing, it is often possible to insure that the prescribed unit stresses shall not be exceeded by properly spacing the eye-bars (Art. 91). Columns and lateral bracing must be arranged with due regard both to economy of shop work and to ease of erection. Field riveting should be reduced to a minimum, since it is more expensive and less satisfactory in regard to strength than shop riveting.

All parts of the metal work of the bridge should be arranged so that they can be easily painted after erection. The shoes,

rollers, and bed plates should be so placed that they cannot become surrounded with dirt from the roadway or approaches. The floor of a highway bridge should be so arranged that water draining from it shall not fall upon the metal work underneath. In short, the designer should endeavor to produce a structure that shall not only be of ample security when erected, but which shall maintain that degree of security through a long life of useful service to the public.

CHAPTER III.

BRIDGE CONTRACTS AND OFFICE WORK.

ART. 13. SPECIFICATIONS.

The local circumstances of the case in hand determine, according to the principles of Chap. II, the number of spans of the bridge to be built, the lengths of the spans, the width of roadway, whether the trusses are to be deck or through, and the character of the traffic. The engineer representing the party that is to own the bridge then prepares rules regarding the loads to be used in the computations, the permissible unit stresses, the quality of the materials, and the character of the workmanship. These rules are called specifications, or sometimes "the specification." All the plans to be submitted by bidders must be in accordance with these specifications, which are afterward made a part of the contract between the buyer and the successful bidder.

Specifications cannot be successfully prepared except by an engineer of experience. In highway bridge work it sometimes happens that county commissioners or town authorities advertise for proposals without having definite specifications, but the result is sure to be that a poor bridge will be erected. Any one can buy a bridge, but only an engineer can do so and obtain both a stable and an economical structure. The highway-bridge specifications of COOPER and those of WADDELL are excellent guides to follow, and they can easily be obtained in pamphlet form. Many railroad companies have their own specifications, and the large bridge companies also have specifications which

they recommend purchasers to follow. The use of such standards by the young engineer will usually result in better work than can be obtained by any specifications prepared by himself.

The following extract from a lecture by THEODORE COOPER states in an excellent manner the fundamental purpose of specifications:

"Their purpose is a twofold one. First: They are to enable bidders upon any work to understand fully the character and extent of the work and what they are expected to furnish and what to do, in order that they may be able to make suitable estimates upon which to formulate an intelligent and proper bid. Second: They are, in connection with the plans, to serve as the reference in regard to all questions as to qualities of the materials and workmanship during the execution of the work, in order to avoid misunderstandings between the engineers and contractors; the contractor not being allowed to furnish poorer or less suitable materials and workmanship than is there specified, nor the engineer to demand any better without giving an extra compensation. Nothing serves better to obtain the best class of contractors and to obviate much of the friction which occurs during construction between the engineer and the contractor than a good specification, carefully and clearly expressed. A loosely drawn and incomplete specification is always attractive to the worst class of contractors, or those who do not intend to do an honest job and who will take advantage of every weak point to get all they can out of the work."

ART. 14. ESTIMATES AND PROPOSALS.

After the preparation of the specifications, proposals or bids are invited from bridge companies for the manufacture and erection of the structure. In general, bridge companies con-

tract for and build only the superstructure, while the piers and abutments are erected by masonry contractors.

The usual mode of procedure is to publish an advertisement which gives the location, length, number of spans, and width of the bridge, stating whether highway or railway, and whether timber, stone, or metal is to be employed. The advertisement mentions where specifications can be seen and information obtained, and names the day and hour when the proposals will be opened. It often states that a certified check for a certain amount must be deposited by each bidder as a guarantee that he will enter into a contract in case the work is awarded to him. Bidders are invited to be present at the opening of the proposals, and the right is reserved to reject any or all bids. It should also be required that each bidder shall present a stress sheet and a general plan of the structure that he purposes to erect.

A bridge company which desires to put in a bid for building the bridge sends one of its agents to the place to procure all the data available. Sometimes the engineer in charge of the work has plans prepared on which the companies estimate and bid, but usually each company prefers to make and submit its own plans. The agent examines closely the locality and estimates the cost of hauling the material from the nearest railroad station, as also the cost of erection. The latter item is often an uncertain one, since delays due to the weather or to floods in streams are liable to arise, and sometimes accidents occur which cause the loss of all profits. It should also be the duty of the agent to become acquainted with the parties who purpose to build the bridge, so that in case of a close competition he may be better prepared to induce them to accept the proposal of the company which he represents.

The computations and designs made by a bidder in order to estimate the cost of a structure are similar to those given in the

preceding and following chapters. The style and proportions of the bridge being decided upon, the stresses are computed by the methods of Part I or Part II, and a stress sheet is prepared, showing these stresses and the sections of the main members. A general drawing is also made showing elevation, plan, and cross-section, with the main features of all details. From this drawing a bill of material is made out, and estimates of the weight and cost of manufacture are prepared. Adding to this the estimated cost of freight and erection, and a fair percentage for interest on invested capital, profit, and contingencies, the bidder decides upon a sum to state in his proposal.

The usual practice in highway-bridge lettings is for each bidder to offer a lump sum for the erection of the superstructure ready for traffic and painted. On railroads it is often the case that the cross-ties, rails, and guard timbers are laid by the railroad company, so that the lump sum is exclusive of the track. On some railroads, however, the proposals are required to be made per pound of the finished structure ready for the track, and in such cases the actual sections of the members are not allowed to exceed by more than 2 or $2\frac{1}{2}$ percent the theoretic sections as required by the stresses and specifications.

ART. 15. LETTINGS AND CONTRACTS.

At the hour stated in the advertisement the proposals are opened and read in the presence of the bidders. The accompanying plans are referred to the engineer in charge to see if they conform to the specifications. It is, however, usually only necessary for him to check the computations of two or three of the lowest bidders if their plans seem otherwise acceptable. On the receipt of the report of the engineer the commissioners or authorities in charge make a formal award of the work to the lowest responsible bidder whose plans are satisfactory, and

he is notified to appear and sign the contract, while the plans and certified checks of the other bidders are returned to them.

It often happens at a bridge letting that the highest bid is about double the lowest. This wide discrepancy is probably due more to the fact that certain companies have better facilities regarding freight and erection than to the relative economy of the several types of trusses. The number of proposals submitted for a structure usually ranges from five to twenty.

This method of bridge lettings, in which each bidder offers his own designs, has many advantages, but it has the disadvantage that only one out of a number of plans is utilized. If twelve bidders each spend $100 in making estimates and designs for a single bridge, there has been expended altogether $1200 which in some way must be paid by the buyers of bridges. It is not an infrequent practice, indeed, that the twelve bidders form a pool, each adding $1200 to his bid, and then the successful bidder pays $100 to each of the eleven unsuccessful ones. This is a necessary evil of the method, perhaps, but the evil is not as great as often assumed, since the expenses of the bridge companies must be paid in some other way if not in this. The expenses of estimating would be lessened if the bidders were limited to plans and designs made by the engineer in charge, but in such cases it usually happens, owing to details of construction, that their bids are higher than for their own designs. Open competition has been one of the elements which has led to the present economic forms of bridge trusses (Chap. II), and, notwithstanding the necessary evils of pools, its results continue in general to be satisfactory.

The contract which is entered into between the parties specifies that the bridge company shall erect the structure according to the plans and specifications, and that the other party shall pay to said company a certain amount for the same. It also sets forth in detail the conditions regarding time of completion

and payment, the liabilities of the contractor for damages due to accidents, penalties for delay of completion, and other conditions mutually agreed upon. When this document is signed both parties are legally bound by its provisions, and the bridge company is ready to begin the detail drawings for the shop work.

A bond is also required to be given by the contractors, signed by them and two responsible bondsmen, binding the contractors under a penalty to execute the contract in pursuance of its terms and conditions, and in accordance with the plans and specifications thereunto annexed. This bond is in law of the nature of a promissory note, and in case of default of the contractors an action at law can be brought to recover the sum stated therein, or such part of it as may be sufficient indemnification for the damages sustained.

There are many engineers who own no bridge works, yet nevertheless bid for and take contracts to erect structures. Such men have arrangements with bridge builders to manufacture their bridges at certain prices per pound, or they make special bargains for the contracts that they secure. Many of these engineers do good work and make a fair profit.

ART. 16. OFFICE PRACTICE.

The engineering department of a bridge company is usually divided into two parts, the estimating or computing division and the detailing or drafting division. The estimating division computes the stresses and makes a stress sheet, giving the principal dimensions and sections, and from this prepares bills of material which enable the amount of its bid to be determined. If it secures the contract, this stress sheet is then turned over to the drafting division, where the details are worked out and the working drawings are prepared. Thus, for the small bridge of

Chap. X, the sheet No. 1 (Fig. 137) is prepared by the estimating division, while the nine other sheets are made by the drafting division. A young graduate on entering the engineering department of a bridge company is generally assigned to the drafting office, where he spends three or four years in obtaining the training that is necessary before he can be promoted to the estimating division.

The working drawings made by the drafting division are for the use of the templet makers, the shop foreman and workmen, and the inspectors. Hence the drawing of each piece should be made so plain and complete that the workmen may clearly and easily understand it. The dimensions of all pieces, rivet spacing, and pitch of rivets should be given in full on the drawings. All printing should be plain and well done, though time should not be wasted in this work. All figures should be large enough to take and show well in the blue print. If the space on the drawings between the rivet heads will not permit of good-sized figures being placed in them, then lines should be projected off to one side of the member and the figures placed between them. Arrow points should be placed at the points between which the distance is given. Fairly heavy lines should be used so as to give a good clear blue print, while fine ones should be avoided except for dimension lines.

The data which the draftsman receives from the computing division consist of the stress sheet showing the stresses in the members and the sections to be used, and a copy of the specifications. The first thing the draftsman generally does is to find out what material is required and how much of it. If the structure is of considerable size, this is best done by laying out the work in a general way on thick brown paper prepared for this purpose, not stopping to put in the details, but going far enough to enable him to determine quite closely what are the lengths and sizes of the angles and plates which are required. He then

consults the list of material in stock, and if he finds any in his bill that is not in stock he makes out an order list, from which the material is ordered immediately by the purchasing department, for it must be on hand as soon as the drawings are finished.

The drawings already laid out in a general way are now completed by placing tracing linen over them and tracing the work from the paper and filling out the details on the tracing linen. In making the details many computations of rivet connections must be made, so that the work shall compare to both the specifications and the practice of the bridge company. The detailer must have a good knowledge of the mechanics of materials in order to be successful in his work.

The tracing is done on the back or unglazed side of the linen. This side shows pencil lines much better than the glazed side, and it will take ink lines just as well. When it is necessary to do any erasing on the tracing linen a rubber ink eraser is used carefully and patiently. The erased area is then rubbed with a stick of pumice stone before inking again, to prevent the ink from spreading. The point of a knife or other sharp tool should not be used to erase lines or spots from tracing linen which have to be inked over again. If the surface of the tracing linen becomes greasy so that the ink will not take well, a little powdered chalk, sprinkled on and rubbed carefully with a cloth, absorbs the grease and gives a better working surface.

After the drawings have been completed a bill of material is prepared. This is made in such a way as to serve as a shipping list also. It contains a list of every individual piece entering into the structure. These are arranged in groups in the list just as the pieces are assembled to make up a member. All pieces which require forging, such as eye-bars, ties, and

counters, are listed also on a sheet known as the forge sheet. Full dimensions and details, and perhaps sketches, are required on this sheet, which then goes to the forge shop. A list of all field rivets and bolts is also made, which gives their size, length, grip, and their location in the bridge.

After the listing has been done both drawings and lists go back to the computing room, where every item, line, and figure is carefully checked. If no errors are found, which is rarely the case, the draftsman may consider his work completed, but if any are found they must be corrected. Blue prints are next taken from the tracings, and the work is ready for the shops.

In the drafting rooms of some of our larger bridge plants there are as many as twenty-five or thirty men, all under the immediate charge of a superintendent or head draftsman, who is thoroughly posted on all kinds of detail work and shop methods. Each man is supposed to be supplied with a complete outfit of drawing tools, and to have a desk to himself, with drawers for paper, tracing linen, and tools. The old style of drawing desk is flat on top and from three feet six inches to four feet in height, with a regular drawing board on top. The more modern desks are not quite so high, and the top is so arranged that it can be tipped up toward the draftsman, making it easier to see and get at all parts of the drawing.

Adjoining the drafting room is a fireproof vault in which are kept all plans and drawings of structures that have been built by the company. These are of great value to the company and also to every draftsman. The vault is the draftsman's library. In consulting it he may find many unique and useful designs and details which will greatly facilitate his work, especially in unusual connections such as occur in skew bridges. A young draftsman should also take advantage of every opportunity to observe and study shop processes in order to be able to see the reasons of the rules of the company regarding bridge

details. The young engineer who does not understand the reasons that govern his work cannot make good and satisfactory drawings for his employer, but he who knows the theory and practice of the subject will do the best work and earn the most rapid promotion.

ART 17. RULES FOR SHOP DRAWINGS.

The following rules for making shop drawings are those given by the American Bridge Company in its Standards for Structural Details, 1901. They are here printed by permission kindly granted by C. C. SCHNEIDER, Vice President. These are general rules applicable to all kinds of detailed drawings; other special rules for drawings of plate-girder bridges, truss bridges, and buildings are also given in the volume above mentioned.

The standard size of sheet shall be 24 by 36 inches, with two border lines $\frac{1}{2}$ and 1 inch from the edge respectively. Small sheets shall be used for beams, pins, eye-bars, etc. Special forms are provided for these sheets.

The title shall be arranged uniformly for each contract near the lower right-hand corner of the sheet. A stamp is provided for the contract, sheet number, etc. It shall be applied in the lower right-hand corner of the sheet. The name of the draftsman in charge of the work shall appear in full, others with initials only.

Detail drawings shall as a rule be made in scale $\frac{3}{4}$ or 1 inch to the foot; for large plate and lattice girders $\frac{1}{2}$ and $\frac{5}{8}$ inch may be used. Larger scales, such as $1\frac{1}{2}$ and 3 inches to the foot, are permissible only for showing certain complicated details or for machine work. Large sheets shall be neatly and carefully made to exact scale.

Members shall be detailed in the position which they occupy in the structure, that is, horizontal members shall be shown

lengthwise, and vertical members crosswise, on the sheet. Inclined members (and vertical ones when necessary on account of space) may be shown lengthwise on the sheet, but then always with their lower end to the left. Avoid notes as much as possible. Where there is the least chance for ambiguity make another view.

Show all elevations, sections, and views in their proper position — looking toward the member. Place the top view directly above and bottom view below the elevation. The bottom view shall always consist of a horizontal section seen from above.

In sectional views the web or gusset plates shall always be blackened. Angles, fillers, etc., shall be cross-hatched, but only when necessary on account of clearness. In a plate girder, for instance, it is not necessary to cross-hatch all the stiffeners and fillers in the bottom view.

Holes for field connections shall always be blackened, and shall, as a rule, be shown in all elevations and sectional views. Rivet heads shall be shown only when necessary; for instance, at the ends of members, around field connections, when countersunk, flattened, etc.

In detailing members which adjoin or connect to others in the structure, part of the latter shall be shown in red, sufficiently to indicate the clearance required or the nature of the connection. Plain building work is exempt from this rule.

When part of one member is detailed same as another, figures for rivet spacing, etc., shall not be repeated; refer to previous sheet or sheets, bearing in mind that these must contain final information. It is not permissible to refer to a sheet, which in turn refers to another. Main dimensions, which are necessary for checking, such as center-to-center distances, story heights, etc., shall be repeated from sheet to sheet.

Holes for field connections must always be located independently, even if figured in connection with shop-rivets; they shall be repeated from sheet to sheet unless they are standard, in which case they shall be identified by a mark and the sheet given on which they are detailed.

A diagram in small scale, showing the relative position of the member in the structure, shall appear on every sheet. The member or members, which are detailed on the sheet, shall be shown in black, and the rest in red, ink. Plain building work is exempt from this rule.

The quality of material, workmanship, size of rivets, etc., shall be specified on every sheet as far as it refers to the sheet itself. Standard workmanship, such as milling and tight fit of stiffeners, milling ends of columns, etc., shall not be specified on drawings.

Each piece which is shipped separately shall have a shipping mark. These marks shall consist of capital letters and numerals, or numerals only; no small letters shall be used except when sub-marking becomes absolutely necessary. The letters R. and L. shall be used only to designate "right" and "left." Never use the word "marked" in abbreviated form in front of the letters, for instance, instead of "3 Floorbeams, mk. G4," say "3 Floorbeams G4."

Pieces which are shipped bolted on to a member shall, as a rule, also have a separate mark in order to identify them should they for some reason or another become detached from the main member. The drawing shall specify which pieces are to be bolted on for shipment, and the necessary bolts shall be billed.

A system of assembling marks shall be established for all small pieces in a structure which repeat themselves in great numbers. These marks shall consist of small letters and

numerals, or numerals only; no capital letters shall be used; avoid prime and sub-marks, such as m'a.

For all lettering use plain letters. For title, main dimensions, and for all marks, particularly shipping marks, use heavy type. Red ink (Winsor & Newton's carmine) shall be used for dimension, reference lines, etc.

Conventional signs for rivets are shown on page 18 [Standards for Structural Details]. Countersunk rivet heads project $\frac{1}{8}''$; if less height of heads is required, drawings shall specify that they are to be chipped or that they must not project more than $\frac{1}{16}''$. Flattened heads project from $\frac{3}{8}''$ to $\frac{7}{16}''$; if less height of heads is required, they shall be countersunk.

Metals in section shall be shown as follows:

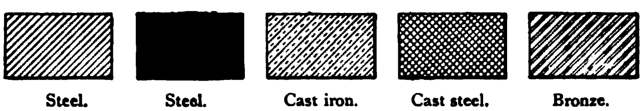

Fig. 12.

Shop bills shall be written on special forms provided for the purpose. When the bills appear on the drawings as well, they shall either be placed close to the member to which they belong or on the right-hand side of the sheet.

When the drawings do not contain any shop bills, these shall be so written that each sheet can have its bills attached to it, if desired; that is, one page of shop bills shall not contain bills for two sheets of drawings.

In large structures, such as elevated railroads, viaducts, etc., which always are subdivided into shipments of suitable size, both mill and shop bills must be written separately for each shipment.

In writing the shop bill, bear in mind that it shall serve as a guide for the laying out and assembling of the member, besides being a list of the material required. For this reason members

which are radically different as to material shall not be bunched in the same shop bill, neither shall pieces which have different marks be bunched in the same item, even if the material is the same.

The main material in a member shall be billed first, followed by the smaller pieces. It is generally a good practice to begin at the left end of a girder, or at the bottom of a post or column. Do not bill first all the angles and then all the flats; when, for instance, the end stiffeners in a girder are billed, the fillers belonging to them shall follow immediately after the angles, and so on. In a column each different bracket shall be billed complete by itself.

When machine-finished surfaces are required, the drawing and the shop bill shall specify the finished width and length of the piece, proper allowance for shearing and planing being made in mill bill. When the metal is to be planed as to thickness, the drawing and shop bill shall specify both the ordered and the finished thickness, for instance, one pl. $12'' \times \frac{13}{16}'' \times 1'6''$ planed to $\frac{3}{4}''$.

Flats and universal plates over $4''$ in width should be ordered in even inches; flats under $4''$ should be ordered by $\frac{1}{2}''$ variation in width. Flats $\frac{1}{4}''$ and under in thickness are very difficult to secure from the mills, and should be avoided if possible.

Every contract embracing different classes of work shall have a subdivision for each class. These subdivisions will be furnished by the chief engineer of the district. Drawings, shop, and shipping bills must be kept separate for each division.

CHAPTER IV.

FABRICATION AND ERECTION.*

ART. 18. ORGANIZATION.

The bridge shop is dependent upon an office. The office stands for organization, which is made efficient by means of four subdivisions: engineering, fabricating, erection, and accounting. The managing head, or superintendent, is the final authority in all matters. His assistants are, for the most part, technically trained men who, as experience fits them, are drawn from the general force. The technical graduate usually begins his career in the drafting room; later he may be transferred to the shop and to the erection forces in the field.

Engineering deals with all structures in their early stages. The chief engineer is responsible for designs, estimates of materials, and details. Since, in the majority of cases, inquiries regarding proposed work are not accompanied by complete plans, stresses must be determined in order to make an accurate estimate of the material required. A knowledge of details, gained in drafting experience, is necessary to make a good designer. The designer does not know when an estimate will result in a contract and must therefore make a careful estimate based on economical design and details. Due to the fact that the mills charge extra for all material over given limits, the summary for the bidding sheet has the material divided on the basis of size, or base price. The amount of material to be subpunched, reamed, or drilled is given, and the physical and chemical requirements are stated because they indicate the

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quality of the desired material. The cost of material is known from market quotations; that of fabrication can readily be fixed from shop cost records; if the work is to be erected, an estimate is made of this cost. This condensed record, or bidding sheet, is always available for comparison with the actual cost on completion of the work. Only a small number of the total estimates made are entered as contracts. They are then turned over to the drafting room where the details are developed as the shop drawings are made. These must be approved by the purchaser's engineer before it is safe to proceed with the work.

Fabrication gives form to the work of the designer and draftsman; it thus appeals strongly to the person who is alert to the growing results of his work. A schedule, or program, giving among other things the particular time when work is to be taken up in the shop, is made monthly for all contracts on hand. This program is based on the contract time of completion, or promise of delivery; it also depends on the completion of the drawings, receipt of material from the mills, and condition of the masonry. It therefore governs the work of drawing, of ordering the material, and of securing this material from the mills. In order to have all main material on hand at the proper time, the work of procuring it from the mills must be systematically followed. Templets are pushed for completion on definite dates, while particular contracts or drawings are ordered into the shop as soon as the templets are completed, or the material has come from the mills. Assembling and riveting are then directed by specific programs.

Some shops are operated under the old daily rate system of payment. Of the newer forms — bonus, premium, and piece-rate system — the fairest is believed to be the premium system. If the worker exceeds the amount of work assumed for the average man, he is entitled to a definite percentage on

this added production; the remainder is the company's share. Once the rate is fixed, it must remain unchanged unless newer machinery results in a decided increase in production. As soon as a drawing is sent to the shop, the time analysis is made. This fixes the time allowed to perform every single operation for each structural member. Hardness of material, speed of machines or drills, amount of handling, and simplicity of details are all considered. From this analysis can be determined the time in which the average man can do the work. When it is being done the actual time of every operation is kept. The record kept of the work of each man shows whether or not he has earned a premium.

As the work is being riveted, or otherwise nearing completion, it is given attention by shop inspectors. They see that plans are followed closely and they are responsible for accurate output. The upkeep of the shop is taken care of by a mechanical staff. Their knowledge and watchfulness control freedom from breakdown and therefore speed of output. Records are absolutely essential. One set has to do with the number of men employed daily in each department. Shipment of finished material as well as that sent to the yard for storage is recorded daily. Each individual drawing has its own record card which gives its actual shop condition at any particular time. Consumption of fuel and power is recorded and compared with output. A record is kept of the life of various tools and their accessories as well as of equipment.

Erection has to do with the completion of the structure in its final location. An engineer has oversight of this division of the work. Many engineering features must be determined in advance. Stresses due to erection often control the main material required and very often the details. Competition may be so keen that the method of erection will secure a contract, if less

costly than one commonly used. A large equipment called plant is necessary to erect bridges due to different conditions prevailing at various sites. To keep this plant efficient and adapted for low cost erection requires the attention of trained men. Every erection operation in the field is a complete business in itself. Payrolls, material accounts, daily reports, and correspondence are handled in both field and home office.

Accounting keeps the managing head in close touch with all the work. It deals with all the costs of operation. Losses, if any, must be shown and guarded against on future work. An accurate list is kept of all material, with its cost, entering into the work of every contract. The shipped weight of finished material when deducted from material shipped from the mills must not show an excessive loss. This is shop waste due to cutting material. Labor cost of fabrication, operation of shop, and overhead cost or management form another class of accounts. The cost of erection is kept well classified and monthly estimates can be made of the cost of completing any contract.

ART. 19. PREPARATION FOR FABRICATION.

In preparing for shop work close attention must be given to four operations: ordering material, specifications and rolling, receiving material, templet and pattern work. Bridge shops associated with the furnaces and mills have the second operation under better control.

Ordering material is done from bill sheets which list every piece called for on a drawing. This material may be plates, structural shapes, forgings, castings, and miscellaneous items. Plates are rolled on a universal mill from 8 to 49 inches in width,—their edges are rolled or finished; plates up to 132 inches wide can be made with sheared or cut edges. Plates for

connections are irregular in shape and when cut at the mill ready for use are known as sketch plates. It is often necessary to consult the mill as to what can be rolled in specific cases. Of the structural shapes angles, beams, and channels are most in demand. Tee bars cannot be readily secured, while zee bars are rolled at few mills on account of little demand. Forgings are used for pins and rollers. Steel castings are sometimes used for rollers, but the latter can be secured more quickly if rolled or forged. Steel castings are rapidly replacing those made of cast iron. Miscellaneous material often requires more information for ordering than most of the others. One of the first things the college man should learn, when in the drafting room, is that material does not come perfectly to the dimensions called for. He should become familiar with the variations allowed in the best rolling mill or foundry practice. As an instance, a girder may have a web 48 inches deep and 50 feet long. With sheared edges the plate may vary as much as ½ inch and, unless the flange angles are set 48½ inches back to back, the plate will project beyond the angles, at some points, unless one edge is planed. In short, unusual sizes or unnecessary refinements must be avoided.

The specifications for making steel adopted by the American Railway Engineering and Maintenance of Way Association are rapidly being accepted by the different railroads for their standard. This uniform specification will simplify mill work and secure quicker deliveries. Naturally when mills are busiest, customers are most urgent in demanding their work. To secure quick deliveries, few sizes of shapes must be specified. The mills should cut out unnecessary sizes such as 4 × 3 and 5 × 3 angles, because 4 × 3½ and 5 × 3½ are better sections and two sections will be furnished more quickly than four. Large highway bridges have as many as sixteen different sections of

angles; to get these rolled at the mills will take two to three months. Standard railroad practice, however, tends toward simplification along this line. When the material is rolled, a number is stamped upon each piece which represents that given the particular heat, or melt, of steel in the furnace. Tests of a specimen from each melt are made and a record kept of its chemical and physical properties.

When the material is received at the shop, it has the mill order number, contract. number, and size painted on it. It should be carefully checked with the ordered dimensions. To discover the errors after material is in the shop is expensive because of delay. If the storage yard is large enough, the ma-

Fig. 13.—Gantry Crane in Storage Yard. Fabricated material, instead of plates and shapes, are stored in yard temporarily.

terial can be separated by contracts. Records show the exact location of all material. For reloading and sending to the shop a minimum time is thus required. Gantry or bridge traveling cranes are most efficient for storage yard service. Several months' material can well be kept in the yard to guard against sudden changes in demands for finished product. Figure 13 is an illustration of a storage yard with gantry crane.

Templets and patterns are the beginning of actual shop operations. These are made as soon as the drawings are approved; in the meantime material is being rolled. A templet, made of white pine lumber or durable paper cardboard, is of the exact size of any separate piece of a structural member, and may be a frame or a single board. In the templet, holes $\frac{1}{2}$ inch diameter in wood or $\frac{5}{8}$ inch diameter in cardboard are cut wherever there is to be a hole or rivet in the finished piece. The cardboard is replacing expensive lumber and both are in part being eliminated by automatic spacing punches. Templet lumber is 4, 6, and 8 inches wide; $\frac{7}{8}$ inch thick; 12, 14, and 16 feet long. Cardboard is 42 by 60 inches and $\frac{1}{8}$ inch thick. All dimensions are taken from steel tapes which are standardized. Since the first essential of good shop work is accurate measurement, all tapes must be true and constantly checked. A draftsman should keep in touch with the templet shop until he knows just what information is necessary or required for a drawing. Unlike the templet, a pattern is a full-size model of the finished casting, but slightly larger. To make it, a shrinkage rule is used instead of the ordinary foot rule. For steel castings the foot actually measures $12\frac{3}{16}$ inches, while for iron castings it measures $12\frac{1}{8}$ inches. Both castings, while cooling, shrink to the standard foot of 12 inches. To know the methods for making irregular castings makes it profitable for the young draftsman to visit the foundry. Slight changes in details of the design often produce better castings. The pattern maker should serve as a check on the drafting room, but usually drawings are finished and approved before he is able to see them to suggest improvements. To protect patterns from the moisture in sand or loam, they are painted with shellac. Pattern and casting work should be kept well in advance of the main structural work, in order to gain time for replacing defective castings.

ART. 20. PREPARATION OF MILL MATERIAL.

For convenience in shipping and to prevent loss, the raw material is received from the mill in long lengths. The main material is delivered cut to size, with an allowance for finishing if required, while the detail pieces are cut from long lengths known as multiple lengths. The structural shapes and the plates should be kept on separate, or opposite, sides of the shop

Fig. 14.—Preparation of Mill Material. Shows separation of plates and shapes, punches in foreground, plate shears in right background, angle shears on left in far background.

for convenience in handling. Broad gauge tracks enable the material to be delivered on railroad cars, from the mill or shop yard directly to the machines in the shop. Overhead traveling bridge cranes and cantilever wall cranes carry the material from

point to point in the shop and finally to the finishing end. Narrow gauge tracks carry the small trucks loaded with short angles, plates, and lattice bars, through the shop to points where the assembling is done. Before the material is ready to be used, much handling is required. It includes the following operations: straightening, marking off, cutting or sawing, bending or crimping, upsetting, buckling, planing plate-edges, milling or chamfering, and finally punching or drilling. Figure 14 shows how plates and shapes are separated in preparation. Punches are shown in foreground, plate shears in right background, and angle shears in left far background.

Since material rolled at the mills distorts somewhat on cooling, the structural shapes must be straightened by running through grooved rolls or on gag presses. Care must be taken in straightening to leave the legs of angles exactly at 90 degrees to secure full bearing for stiffener angles of plate girders. Plates are straightened by running them through straightening rolls consisting of six or eight sets of parallel rolls. Loading, unloading, or sorting material, especially plates, results in bending or kinking due to improper handling appliances. Before they can be used in the shop, the plates must be straightened by running them through rolls. Plates become curved from irregular punching; they may be straightened by increasing the pressure on the one side of the plate.

When the material is straight and flat, it is in proper condition for marking off. The templet is laid on the metal piece and securely clamped. A special center punch, placed in the templet hole and given a blow with the hammer, leaves a conical-shaped indentation in the material. The correct length is marked by scratching a line at the end of the templet. This is then made more distinct by small center punch marks which give it the appearance of a dotted line. All the punching, cut-

ting, or any other work to be done is indicated on each piece just as called for on the drawing. Material to be bent is marked off after the bending has been done. Pieces for small details or short lengths are first cut to exact size and then marked off and punched. The contract number, drawing number, and assembling, or indentification, mark are painted on each large piece, but on only a few of the duplicate pieces when there are many of them. It is known, therefore, for what particular member this material will be required.

The machines for cutting or sawing are close to the end of the shop where the material is delivered and marked off. The bulk of the material can be cut or sheared. The angle shears can make square and beveled cuts; it has two cutting blades, symmetrically placed, in order to cut angles in pairs called rights and lefts as shown in the accompanying diagram.

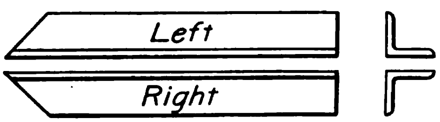


Fig. 15.

The beginner usually has trouble to understand right and left, or opposite hand. If he conceives of an end post, with portal connection angles on the inside web, or a shoe with bottom lateral connection plate, placed before a mirror, he will see the left, or opposite hand, reflected in the mirror; the object itself is the right or "as shown." In other words, the object and its reflection are symmetrical about the plane of the mirror. Being different in at least one or two points, which if omitted would leave the objects alike, the one cannot be substituted for the other by reversing or turning in any manner. Just as objects are paired, so small pieces are paired. Beams and channels are cut on special machines, so also the coping is done when part of the flange and web is removed that the top or

bottom may be flush when one is connected to the other. If finish is not particular, channels can be cut on a plate shear; otherwise they must be sawed like beams. Plate shears consist of a straight knife, or shear blade, capable of cutting a plate 132 inches wide and 1 inch thick, down to smaller shears of the same type and also of a different type called the alligator or splitting shears. Plates sheared at the mill overrun their dimensions frequently and must be sheared to the nominal size called for on the drawings. Lattice bars are cut from long bars which have the holes and two round ends punched with one stroke of the machine. The saw known as the cold saw, circular in shape, runs at a comparatively slow speed and mills off small chips as it cuts through the section.

Bending or crimping is done before the holes are punched. When there is little duplication of pieces, the bending is done by a blacksmith. The angles and plates are heated at the point to be bent; then one part is fastened in a clamp or on a flat bed and bent to long or short radii by hammering. Heavy sections or duplicate parts are bent on the hydraulic press with the aid of dies or bending forms. Curved end angles of through plate girders and curved or angular knee braces are thus bent in an economical manner. When the radius of the curve of any material is large, it is bent cold on a bulldozer. This machine bends material in small arcs by exerting light or heavy pressure between two fixed supports, thus producing in a beam a small or large permanent deflection between the supports. Thus the small divisions of the piece are so bent that the result is a uniform curve. In order to save the weight of fillers, stiffener angles are crimped so that they can fit over flange angles and plates. This crimping is rapidly and uniformly done on the hydraulic press, after the angles have been properly heated.

Upsetting is necessary to form eyebars or rods having upset screw ends. Rods are upset so that, when a screw thread is cut in the upset end, the area at the root of the thread will be slightly greater than the area of the bar itself, since rupture should not occur in the screw end. The threads are cut on the rods for nuts, clevises, turnbuckles, and sleeve nuts. Eyebar flats are upset to form a round head in which a hole is punched and then bored for a pin. This head is made large enough to have the section normal to the axis, through the finished pin hole, 33 to 40 percent larger than the section of the bar itself. After eyebars are forged, they are annealed to remove the initial or unequal stress between the head and the body of the bar or at the junction of heated and unheated material. Specifications require that all eyebars in the same panel, when piled on top of each other, must permit the pins at each end to pass through the pin holes without forcing. The bars, after annealing, must therefore be bored very accurately to standard tapes and at a uniform temperature.

When solid steel floors are required for highway bridge work, buckle plates are used. They are also used for lock gates of bridge weirs or whenever it is not desirable to use angles to stiffen a plate. Buckling is accomplished, without heating the plates, by pressing a shallow pyramid into the original flat plate. Before buckling, the sides of the plate are parallel; after buckling, they are pulled in about ½ an inch opposite the deep part of the buckle. But between the buckles the plate is its original width. If a uniform edge distance must be maintained, wider plates should be ordered and resheared after buckling; but this adds to the cost. On account of the distortion of the edges, punching must be done after the buckling.

Some specifications call for sheared plate edges to be planed, especially when exceeding a given thickness, or when used in

certain members. Planed edges are confined principally to wide plates and gusset or connecting plates of riveted spans. The plate is clamped in a flat position; the cutting tool moves along the edge to be planed.

Stiffener angles of plate girders must all be of the same length, especially if they are to bear against flange angles. To secure this result and also to have the ends square, they are milled in a special machine for this purpose. Furthermore when a piece fits tightly into the root of an angle it must clear the fillet. The end of the stiffener angle must therefore be rounded off, or chamfered, for bearing; this is done on two angles, back to back, at one operation.

Punching and drilling complete the preparation of the material. A punched hole is not a cylinder, because one end is the diameter of the punch and the other is the diameter of the die. The punch is always smaller than the die, as in Fig. 16. Specifications state that they shall not differ by more than $\frac{1}{8}$ inch and that the diameter of the punch is not to be more than $\frac{1}{16}$ inch greater than the diameter of the rivet. To make this a better hole in thick material, subpunching was resorted to; now the original hole is $\frac{1}{4}$ inch small and reamed to size. When several pieces are assembled on each other, they do not have a common center line because it is impossible to punch exactly on the center punch marks. In such cases, subpunching and reaming after assembling correct these irregular holes. Heavy structures require heavy sections and a large proportion of reamed or drilled holes, thus forcing a lower drilling cost. The advent of high speed tool steel helped to bring about a

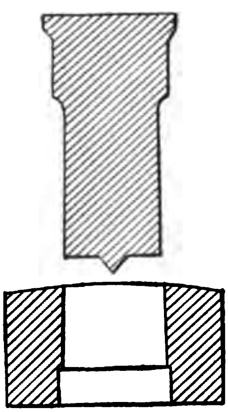


Fig. 16.

larger proportion of drilling, so that, at the present time, completed members are assembled and have their splice connection holes drilled from the solid. To secure low cost punching, multiple and automatic punching machines are used when there are many duplicate parts, as in bridges having several spans of the same length, or elevated railroads and viaducts. These machines are of the simple type, rack-adjustment type, and the complex type which is controlled by a paper templet roll similar to a music roll of a mechanical piano player. Holes punched on such a machine will match, because the same templet is used in all cases to punch the webs, then all flange angles in pairs, and finally the cover plates.

The minimum work done on any piece is cutting, or sawing, and punching when the multiple punch is used to eliminate templets. The maximum number of operations is six: straightening, marking off, cutting or sawing, bending or crimping, milling or chamfering, and finally punching or drilling.

ART. 21. ASSEMBLING MATERIAL AND RIVETING.

The work of assembling and riveting includes the following operations: accumulating material; assembling simple members and units of compound members; reaming after assembling; riveting simple members and compound units; finally assembling, reaming, and riveting compound members, or assembling and riveting complete spans. The output of the shop is limited by its floor layout, tool equipment, and method of handling the work. Shops with high central stories are well adapted to this work, because they admit much light and provide a place for the operation of several types of cranes. The overhead traveling bridge crane consists of two parallel box plate girders mounted on trucks. On the girders is operated a trolley for transverse shop movements, the trolley being equipped with two or more

vertical hoists for light rapid lifts, or slow heavy lifts, of 15 to 65 tons capacity. These cranes are operated by a man in a cab attached to the crane. Underneath this crane are wall cranes. The cantilever wall crane, which is also operated by a man on the crane, moves up and down the shop and, at the same time, has a trolley moving out on the projecting cantilever arm toward the center of the shop. The capacity is 5 to 10 tons and the lift is rapid. Other cantilever wall cranes are operated from the floor by the particular gang using it. The material is delivered to the assemblers on narrow gauge trucks and is afterwards moved through the shop, by the crane, over the heads of the men. Permanent skids, about 18 inches high and properly spaced on the floor, serve as assembling benches. Large plate girders can be assembled in upright positions, and to prevent overturning have their flanges bolted to keel blocks in the floor.

The accumulation of material is made easier if a storage place is provided in the shop for the small material as it is being prepared and while waiting for that requiring a large number of operations. This space should be located in the main shop commanded by shop cranes, and may be enlarged by an extension on each side of the main shop. These small pieces include lattice bars and angles; tie, batten, and pin plates; diaphragm plates and angles; and connection angles. When all the detail pieces are prepared, the main material can be started on its course through the shop.

When assembling is begun on any piece, all the material is delivered to a gang of assemblers. The surfaces in contact, because they will be covered up, are given a coat of the kind of paint specified, and the piece is then bolted up tight with sufficient bolts and drift pins to secure all parts and permit handling. Webs, or covers of compound members, are assembled as simple pieces. At the same time, the diaphragms or brace

frames belonging to them are also assembled. At this time, the great advantage of assembling marks will be realized and appreciated for securing the rapid assembling of the material to conform exactly to the drawing.

Since the irregular holes of assembled pieces, referred to in Art. 20, must be corrected before riveting, reaming is the next operation. This part of the shop must be equipped for rapid work, so that the riveters will not be compelled to wait for

Fig. 17.— Reaming Shop. Radial reamers are located along the sides. The assembling shop is adjacent in the background.

material while such a large number of holes must be reamed. Multiple reamers are used as well as the single type, an efficient form of which feeds the drill by compressed air, instead of screw feed, while the drill is operated by an electric motor. Twist

drills without lubricant are now specified, for the work of reaming. This change proves to be an advantage on account of high speed steel being used for the drills. The diameter of the holes before reaming is specified $\frac{1}{4}$ inch small. Unless the material is tightly bolted, chips are forced between the surfaces; this foreign material will prevent rivets from being tight. Figure 17 is an illustration of a reaming shop.

Riveting is rapidly done so far as the actual upsetting of the rivet heads is concerned, but a large time element is consumed in delivering and removing material and preparation for driving rivets. For plate girder work, massive, powerful, hydraulic riveters are mounted in the floor of the shop with a pit in which the girder is moved to and fro and raised up and down while rivets are being driven. Another method is to bring the tools to the work. Portable riveters are of the single and double toggle type, operated by compressed air, and the hydropneumatic type, which operates by a compressed air cylinder acting on an oil cylinder. The single toggle riveter is used for light work; for heavy work, requiring long rivets of large diameter, the double toggle exerts a pressure of 60 tons or more. Both are very efficient.

The pieces to be riveted are supported on high or low trestles, while the riveters are moved by a wall crane operated by the riveting gang from the floor. When not in use the riveter can be hung on wall brackets. Heating furnaces, using coal, oil, or gas for fuel, are placed near the work, but care must be taken not to heat rivets too hot or too long. Rivets are rolled from a soft grade of steel, low in sulphur and phosphorus, into rods $\frac{1}{64}$ inch less than the nominal diameter of the rivet. These rods are upset in a special machine to form a rivet head; at the same time the rivets are cut to required length with sufficient material left, in the shank, to form a second head when driven

in the shop or field. Plates riveted to beam flanges or channel flanges are not parallel to the beveled flanges and therefore make riveting troublesome. Unless the holding-on tool of the riveter is the exact bevel of the flange, the adjusting screw of the snap will be bent. Figure 18 is an illustration of one side of a riveting shop.

Fig. 18.—Side of Riveting Shop. Shows floor-operated cranes, hydropneumatic riveter suspended in foreground, toggle riveter farther back, and rotary planer in near background.

After webs of compound members, such as box girders and compression members with two to four webs, have been riveted, they are returned to the assembling shop, and are then put together as a whole with diaphragms, covers, and lacing bolted on. When drawings call for distances back to back, or limiting dimensions, not more or not less, then compression members must have their webs placed as accurately as possible. To

avoid overrunning or underrunning, wooden blocks of exact length are placed between the webs and held securely by bolting the webs. The holes are reamed and the rivets are finally driven, but in some cases the space is so restricted that hand tools must be used for reaming and riveting. Care must be used in riveting up all pieces, so that there will be no warp, called wind, which is especially liable to occur when there is lacing on two or four sides. For convenience in erection, it is often advantageous to rivet up in the shop, complete, deck plate girder spans, viaduct bents, and turntables. Before the assembling and the riveting of complicated or unusual sections are attempted, a study is made of the different methods of riveting for closing up the section. This is necessary in order that the greatest number of rivets, or the rivets closing a box section, will be driven with the power riveters instead of the hand riveters. The order of riveting the component parts therefore fixes the order of assembling.

ART. 22. FINISHING AND INSPECTION.

The work of finishing and inspection requires a definite amount of shop space in order to be carried on expeditiously. Finishing is concerned with small and large operations and will be considered in the following order: final operations other than riveting, erecting movable parts, assembling complete trusses, and the work done in the auxiliary shop and machine shop. Inspection has to do with the shop inspector and the foreign inspector.

The final operations are very elaborate in some cases, but this is not the rule with the bulk of the output. Milling or facing is necessary to secure an even bearing surface, to provide a finish, or to prevent overrun. This work is done on a rotary planer, which consists of a rotating disk provided with holes for setting

the cutting tools, and a table on which to clamp the work. The head with its disk can be revolved horizontally to make a cut at any angle. This machine is shown in Fig. 19. Floor beams,

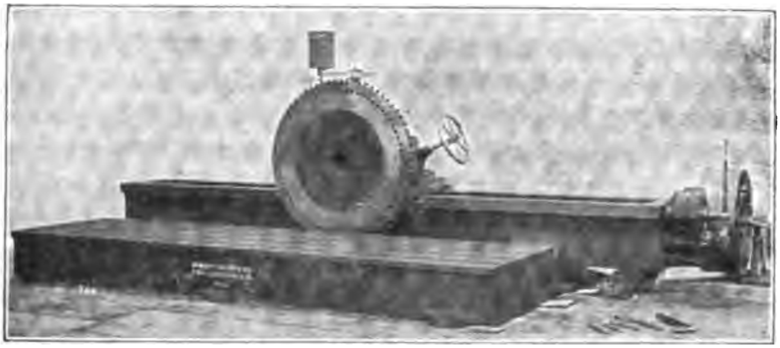


Fig. 19.— Rotary Planer.

stringers, chords, and columns are the members most frequently faced. Pinholes are bored on special machines, which should consist of two boring heads mounted on one base. Since distances, center to center, of pinholes can be set and checked on the machine, the result is accurate work; and, with only one setting, duplicate members can be made exactly alike. Rolling bascule bridges require segmental girders with heavy base plates for safe bearing pressures. The webs and the flange angles as well as both sides of the base plate are milled to a true circumference on a special machine. Stringer and floor-beam connection angles, as well as truss connections, are reamed to metal templets made of $\frac{3}{8}$-inch plates carefully laid out and drilled to take a steel thimble for keeping the drill normal to the surface. Spliced members are assembled with all splice plates in position and have all holes reamed or drilled from the solid. Each loose piece is marked by an assumed letter or number common to that particular splice. Splice plates must be well bolted to prevent loose chips from wedging

between plates or angles. Figure 20 is an illustration of a finishing shop.

Fig. 20. — Finishing Shop. Shows floor space for working with hand tools and reaming to templet, portable drills for splice and connection holes, pin-boring machine. Riveting shop is adjacent in background.

Erecting movable parts is necessary to correct errors which would not be discovered until field erection was under way. Turntable centers are assembled in the machine shop, where a test is made of the number of pounds pulled by a spring balance at a given distance from the center of the turntable, sufficient to start turning. The complete track, drums, and turning shafts of rim-bearing swing bridge spans are assembled, and the whole turned by hand, before shipment is made. All connections are reamed, accurate bearing secured for the wheels, errors corrected, and the whole carefully matchmarked. The center line is

scribed on the track plate and the actual diameter should be stamped on the face, so that erectors can start with the actual diameter, thus securing properly bolted joints. It is not possible to get the actual diameter shown on the drawing on account of facing, radially, the ends of the large number of segments.

Heavy riveted trusses are assembled complete, under a crane, and all connection holes reamed with large portable reamers. Not only are the connections then carefully matchmarked, but also a diagram is made to give the erector in the field the marks used and their location. The burrs, formed by the drills, are removed with a small air burring tool, which makes a fillet of about $\frac{1}{16}$ inch on the edge of the hole. An assembled truss is shown in Fig. 21. The scale can be determined from the fact that the crane is 85 feet, centers of legs, and the truss 157 feet.

Fig. 21. — Reaming Assembled Truss. 280-foot deck riveted truss for I. & W. N. R. R. bridge over Pend d'Oreille river at Irene, Wash. Gantry crane for assembling and loading.

An auxiliary shop can be used to great advantage in connection with the main shop. Heavy work is moved through the latter, while the former will take the light work, such as lateral bracing, sway bracing, truss work, knee braces, shoes, pedestals,

and grillages. It should be equipped with a heating furnace, portable and stationary riveters, a portable drill, and a planer. The material is delivered from the punch shop on narrow gauge trucks, but to facilitate weighing and shipment, the auxiliary shop should be close to the finishing end of the main shop. A complete machine shop equipment is necessary, not only to keep shop tools and machinery in repair, but also to take care of all contract work requiring machine finish. Pins, roller nests, and shoe castings for girder and truss bridges are finished here, as well as complete operating machinery for swing and bascule bridges. In connection with the machine shop, a blacksmith shop is required to make tools and forgings and do the repair work for the shop. Heavy power hammers are indispensable for the latter, as well as jib cranes to command the fires or heating furnaces.

Shop inspection applies more particularly to the finished product. The shop inspector, with his assistants, is responsible for all work fabricated; it cannot be shipped until he has approved. He must examine the work to see that it conforms to the drawing, that it is correct in all its limiting dimensions in order that errors may be corrected with shop tools instead of causing expensive work in the field.

The purchaser's representative is known as the foreign inspector. He takes care only of that part of the shop's output for which he was engaged. He is employed by the purchaser or is the representative of an engineering or inspecting firm which has contracted to do the inspecting at a given price per ton. If the tonnage is not large, he will handle the mill as well as the shop inspection. He is not always required to witness the rolling of material but, having been notified as soon as test specimens are prepared, he must be present when the tests are pulled. He accepts material for which the tests have passed

the chemical and physical requirements, and examines rolled material for surface defects or flaws. When full-size tests are made on finished members, such as eyebars, he is also present. At the shop he keeps in touch with the handling and punching of the material, so that it is not damaged by carelessness. He sees that inaccessible parts are painted, that rivets are tight, and that the character of the workmanship complies with the specifications. He must be sure that sections or thicknesses agree with the drawings and that clearances are maintained; if the material is erected by the purchaser, he checks all connections and principal dimensions. When satisfied that the finished piece is correct, he stamps his private mark on the metal with a circle of paint for readily finding the mark.

ART. 23. PAINTING AND SHIPPING.

The material, having been completed in conformity to the drawings, can then be painted and is ready for shipment. Specifications usually stipulate one coat of shop paint and two field coats if erected under contract. All machined surfaces are to be coated with white lead and tallow. In the process of manufacture, not only does the material not become freed from mill scale, but grease and oil from cranes, reaming and riveting machinery adhere to the metal. Chips produced by drilling and milling catch in narrow openings and corners, or stick to excess assembling paint as it dries. Specifications state that all this shall be removed, and also that no paint shall be applied to wet surfaces or in freezing weather. The shipping mark should not be painted on the bare metal but on the shop coat when dry. The contract and drawing number are also painted with the shipping, or erection, mark to provide full identification in the field. Specifications, in giving the number of coats of paint to be applied, sometimes state that the

paint is to be subject to the approval of the engineer. In bidding, it is necessary to know whether lead, mineral, or carbon paint is to be used, because the first mentioned is the most expensive and three coats make quite a factor in the cost.

Painting is disagreeable work at best and foreign labor is the only kind that can be held to it. Constant supervision is necessary to see that scale, oil, and chips are removed, because the workman reasons that if everything is covered with paint nothing more could be desired. In painting corners or inside spaces of complicated members the painter cannot help but get the fresh paint on himself, therefore he will not look closely, but reaches in and daubs at corners.

The primary object of paint is to afford protection for the metal and not to be simply a covering. On account of the guarantee given by some manufacturers for special brands of paint, they furnish an inspector for the painting to see that the first application is a preservative coating. He is, therefore, very particular to have all foreign material removed, awkward or interior corners properly covered, and any slighted surfaces corrected so that the paint is not too thick, and the whole covering uniform.

A standard specification calls for the shop coat to be of linseed oil, and the inaccessible surfaces, which are covered up in the field by riveting surfaces together, to be covered with a coat of red lead. There is no opportunity to apply the latter coat properly in the field. The oil coat, being transparent, allows any foreign substance to be seen, and it can be removed before applying the field paint. The shop coat is often cut through or rubbed off in spots during shipment and rusting begins, especially if considerable time elapses before erection. The rust must also be removed properly before applying the field coats. The two field coats are generally of different colors,

so that it will be known at all times whether two coats have been applied. The records of previous work show how many gallons of paint per ton of metal are required for each coat on different classes of structures.

Shipping requires as careful attention as all preceding operations. The output of the shop is constant, covers many contracts, and can be filled by a few heavy pieces or very many light-weight pieces. The difficulty of the task can be realized when trying to ship in carloads to secure minimum freight rates; to ship by separate contracts; in definite sequence to meet erection requirements; to save demurrage charges on railroad cars while loading; and to keep the painting shop free from congestion. Every individual member is weighed before shipment, some specifications requiring this to be done before painting, and sometimes requiring the inspector to be present at the weighing.

As soon as material is weighed and loaded, an invoice is written giving the mark of each member and its weight. Payment for material is based on this scale weight, which must not exceed by more than two percent the weight estimated from the drawing. This is known as the pound price contract; but there is no limitation in the lump sum contract, for which a definite sum is paid irrespective of weight. Inspection is paid for on the tonnage rolled and inspected at the mills. This weight exceeds the manufactured, or shipped, weight; the difference between mill shipments and shop shipments is known as shop loss and is due to cutting and machining. This loss ranges from three to ten percent and is excessive when over five percent.

Shipping rules are very explicit, and no car is accepted by a railroad company until their inspector is satisfied that the material is properly secured and comes within all clearance lines. A quarterly publication called Railway Line Clearances gives the M. C. B. Rules Governing Loading and Carrying Structural

Material; Girders; Turntables; also Maximum Clearance and Weight Tables. These M. C. B. rules, formulated by the Master Car Builders, a coöperative association of the different railroads, are fully illustrated with sketches showing the manner of loading and the size of all material required for that purpose. These rules are necessarily stringent, as they are based on experience gained from wrecks and car troubles.

Special requirements develop in the loading, routing, and delivery of material. When pieces of unusual size or weight result from certain designs or types of structures, outline sketches are made and submitted to the railroad authorities. They then advise how the material is to be loaded, give the class of car, and the necessary clearances to be maintained. When the loading is beyond the limits of the equipment, a special car will be built, if tonnage warrants, as in the case of Blackwell's Island Bridge. If the bulk only is unusual, the railroad company designates the special routing by which it is to be consigned. The Official Railway Equipment Register, published monthly in New York City, gives the classes of cars, numbers, and capacities for all railroads. This information is indispensable for the routine of shipping. When material must be rushed to destination, the tracing of cars is necessary. A man sent to follow the shipment keeps in touch with the yardmaster at all freight division points or junction points. He must get his car or cars placed in a preference freight train, or fast freight; but, should any defect develop in a car, it will be set out of the train on the first siding. This siding may be many miles away from a yard, with its repair facilities, and so cause delay which might have been saved by foresight. A car too heavily loaded has had as many as three journal brasses burned out in going a distance of two hundred miles, with a consequent danger of injuring the axle journal proper.

ART. 24. PREPARATION FOR ERECTION.

When the contract calls for the erection of the structure, as well as fabrication, a large amount of additional work is required. In preparing for the erection, it is essential to consider every element that has to do with this part of the work. The phases to be considered are contract requirements, condition at the site, plant requirements, office planning, and the building of special appliances. Large railroad systems do not have work erected by contract, but have it done by their own forces. Their renewal of old structures and new construction work makes possible continuous and economical erection and so justifies full equipment for light and heavy work.

The contract requirements are very important because of their legal status; a penalty is frequently attached for non-completion within a specified time. The time of completion should be based on the furnishing of complete information to permit the drawings to be finished, the material to be ordered, and the shop work to proceed; it should also be based on the condition of the masonry, the most fruitful cause of delay. Structures are frequently projected or wanted in such a hurry that the time of erection for normal conditions only is assumed. The season of the year and flood conditions combined may confine the actual time of erection to a short period in which the erection must be speeded to reduce the risk. The majority of cases now require traffic to be maintained for the railroads using the structures; this practically doubles the trouble and cost of erection. Since in the case of navigable streams there is a short period in the winter when boat traffic is discontinued, freedom from difficulties due to that source is given. Unfavorable conditions may cause delays of a year or more, in which case a large amount of raw or finished material may be tied up. Provision should therefore be made in the contract for percent-

age payments after material is delivered at the shop, after fabrication, after delivery at the site, after erection, and finally after completion of riveting and painting.

The condition at the site must be fully known before it is possible to plan for erection. In new construction, the masonry is very uncertain and generally is completed long after the time named when the contract was awarded. Overhead wires will often interfere with the placing and the operation of derricks and thus block economic erection possible at that particular site. Material may be picked from cars on the main track, or by using the old structure overhead. Falsework may or may not be required to carry traffic, or to support the old structure. The profile of a stream underneath the bridge may be secured from the engineer at the site, or personal examination and measurements may be necessary to decide the character of the falsework. Several trips may be required to keep in touch with changing conditions and to locate tracks and sidings for delivering and erecting the steel. When the site is at a distance from a town or village, arrangements must be made to provide living accommodations for the bridgemen.

The plant requirements divide themselves into temporary and permanent. The former can usually be secured locally. A building is necessary in which tools can be stored and a small office provided. The size of the job often necessitates having a small blacksmith shop to keep the tools in repair. When the temporary framework requires long timber for the traveler and falsework, it usually must come from the mills or a previous job instead of being secured locally. The permanent plant equipment must not entail too heavy a capital investment. The ideal conditions would be to have tools released from one job to be used immediately on the next and to have just enough so that they would always be in use. Plans are continually

made to use, on a particular job, certain hoisting engines and other plant equipment as soon as released from a similar preceding job. The actual conditions are changing so continually, however, that no particular plant can be depended on, but when the masonry is ready the first available one must be used.

The office planning is of more importance than was realized by erectors several years ago. This is due in great part to the fact that traffic must be maintained. It has been a custom to send out the erector with a definite number of tools and, when he reached the site, the method of erection would be decided during the first day. The chief essential to keep in mind is rapid erection to avoid loss from water or storm; it depends largely on the position of splices and the weight of material. The nature of the connections determine how the structure is to be put together and how quickly individual pieces can be released from the hoisting tackle. Excessive weight for a few pieces should be avoided, and the bulk of the material should be uniformly of a weight consistent with the type of structure, because weights of pieces control the size and power of engines and tools. For this reason, erection considerations should control the design, and should not be hampered by unusual conditions. The second essential is low cost erection. These two requirements fix the scheme of erection, and it cannot be determined until a careful study has been made of the conditions. It is frequently necessary to develop several schemes and estimate the cost of each in order to secure the lowest cost. The whole is subordinate to the safety of the men, and the traffic passing the structure, for all operations during erection. Complete details are drawn up for falsework and travelers, especially for special appliances, which should have full instructions provided to render them fool proof. For large structures, diagrams are made giving the weights to be lifted, number of different diameter rivets to

be driven, amount of paint to be applied, points to be watched when working close to limits, as well as general instructions on the method of procedure.

It is becoming necessary to prepare erection travelers and framework at the shop, especially since steel is used to make the appliances which must do the continuous heavy lifting. The power tools of the shop, yard cranes for handling, and ample yard laying-out space, which usually is not available in the field, secure rapid and economical framing. Duplicate operations compel the building of special appliances to reduce cost of plant and of erection. These appliances pay their cost on one erection job if there is sufficient duplicate work. Derricks have long been undergoing standardization in wood and steel; the same methods are now being applied to gallows frames, travelers, cars, and especially to the innumerable small tools necessary for the erector.

ART. 25. SIMPLE ERECTION.

Simple erection has to do with light and short structures which do not require elaborate or expensive plant equipment. Beginning with simple schemes, it develops into larger and heavier erection which requires the same type of appliances. It can be classified under four heads: crude erection, gin pole erection, derrick erection, and gallows frame erection.

Crude erection might be termed primitive erection and in many cases it proves to be ingenious. This must sometimes be resorted to when structures are light or a long freight haul makes the charge on power tools prohibitive. As a rule, average bulk or weight is not moved through much vertical or linear distance and the only tools required are jacks, bars, rollers, and hoisting or pulling tackle. Deck plate girder spans can be assembled and riveted upon the cars used to ship them, jacked up to allow the cars to be pulled out from under, and then lowered

into position; or be slid off sidewise from the cars on a crib-work of railroad ties and then jacked down into position. An erector caught with insufficient equipment, or in an emergency, often develops effective methods which can be applied regularly.

When many pieces must be lifted, or the height for raising becomes considerable, the gin pole is resorted to. This is simply a long pole, or mast, with the bottom rounded and seated on a suitable base to distribute its load, while the top is held by guys of manilla or wire rope. These guys are fastened to a spider attached to the top of the pole; the free end of the guys is secured to temporary or permanent anchorages. When the pole is moved from one position to another, the bottom is pushed with bars or pulled with tackle; in the meantime it is kept vertical by tightening, or taking up, the guys in front and at the same time loosening, or slackening out, those in the rear. Gin pole erection was the original method used for erecting plate girders; the girder was picked up at its middle and raised or lowered into position. In erecting light buildings the pole is moved along the sides to erect columns, while roof trusses are being assembled flat on the ground and riveted. It finally is moved down the center of the building and places the trusses on the columns. For small work, a short pole is mounted on a sill forming a long base to which it is braced, on each side, by diagonal struts; it leans toward the piece to be lifted and is held at the top by one guy or a set of tackle which takes the tension when the weight is lifted. Two of the poles erect the columns and a gin pole raises trusses when rapid erection is wanted.

The derrick is a very handy erection tool commanding any position within reach of its boom, while the gin pole must be moved for every lift it makes. A derrick consists of a vertical mast and inclined boom hinged at the bottom of the mast. The top of the boom is connected to the top of the mast by a set of

tackle; the load hanging from the top of the boom, by means of another set of tackle. The top of the mast may be held in position by guys, as in the case of a gin pole; or by a stiff leg and two guys in a plane normal to the stiff leg and mast, but extending in opposite directions from the top of the mast; or by means of two stiff legs only. The stiff legs are stiff diagonal members extending from the top of the mast to the end of a horizontal stiff member, called the sill, which ties the bottom of the stiff leg to the bottom of the mast. The sills make an angle with each other varying from 60 to 90 degrees, while the stiff legs are inclined at an angle of about 45 degrees. A derrick is therefore known as a guy derrick, combination derrick, or stiff leg derrick. A hoisting engine furnishes the power for operating the derrick. In the case of a large bridge or many spans, one or more derricks are necessary to unload the material for storage until required in the erection. For erection purposes, a derrick is set up near the abutment of a bridge and picks its load from the main track or adjoining track and places it in position. Deck plate girder spans riveted up complete are handled in this manner, as well as heavy single deck or through plate girders. The derrick is especially handy for placing the numerous pieces of the floor system and bracing of the through span. When there are several adjacent girder spans, the derrick can be moved out on top of a girder, the sill and stiff leg being in the same plane as the girder; the mast is held by guys up and down stream. The girders to be erected are then picked up from the span on which the derrick rests and moved forward into position by letting out, or lowering, the boom; the rear end of the sill having been clamped to the girder on which it rests and which serves as counterweight. When traffic is to be maintained, overhead truss spans are used to swing new deck plate girder spans into position, and the derrick on the abutment removes the old truss. In the case of deck or through riveted

truss spans, the bottom chord and floor system are erected on falsework support. The web members are then bolted to the bottom chord and finally the top chord is lifted into position and the top lateral bracing is put in place. If traffic is to be maintained, there should be a derrick at each end of the bridge, for rapid work when placing the floor, particularly for long spans. A pin-connected truss is hard to erect with derrick only, because all members are not stiff and also because of the number of pieces to be supported until the pins are driven.

Fig. 22. — Riveted Truss Span Erection. Shows gallows frame principle and sliding old span sidewise, traffic being maintained. Lehigh & New England R. R. bridge over Delaware river at Portland, Pa.

For lifting only, when no backward or forward movement of consequence is required, the gallows frame is very suitable. This consists of two vertical posts surmounted by a cap composed of two timbers placed edge up having a space of about six inches between. Through this opening hang the chains or rope lashing for holding the tackle blocks. The cap projects

beyond the tops of the legs so that diagonal braces may tie the end of the cap to the bottom of the leg. Such a frame is placed on the masonry, at each end of a span, high and wide enough to clear traffic, and is well braced parallel to the tracks. When a deck plate girder span is to be replaced, the new span is lifted from the cars by a set of tackle at each of the four corners and the cars moved from under the span. The rails are pulled ahead on the track, old ties are thrown below, and old bracing dropped from between the old girders which have been moved sidewise away from the track and braced by shores. When the new span with the new ties on top is lowered into place and the rails have been replaced, traffic can again be resumed. All this work must be done during the longest interval between passenger trains. The old girders are then picked up by the gallows frame and loaded on cars. This method has also been used for deck truss riveted spans, but the truss spans had to be moved sidewise to clear the new trusses. See Fig. 22.

ART. 26. DERRICK CAR ERECTION.

A large item of erection expense is the handling and transportation of plant, and the time consumed in the erection and removal of temporary lifting appliances. This is largely overcome by the use of the derrick car, an ideal erection tool, especially when weights are not excessive and structures are properly detailed as to splice locations. This article will therefore treat of movable derricks in the following order: derrick cars, derrick car erection of plate girder spans, riveted truss spans, pin-connected spans, and viaducts, as well as the use of locomotive cranes, wrecking cranes, and lighters.

A derrick car is essentially a flat car with a derrick and engine mounted on it. The mast, or A-frame, and boom are at the front end of the car; they are braced to the rear end with

tension ties or stiff legs. The body of the car serves as a sill. At the rear end of the car is placed the boiler and engine to furnish power for lifting and for propelling the car by direct gearing or chain drive. A separate propelling engine gives more effective control. The mast of a car derrick must be short to clear overhead obstructions and to keep the car's center of gravity as low as possible. Because of this low mast, the stress in the derrick members is practically double that of the normal derrick. The requirements of a derrick car are that it should be self-propelling, thus being independent of costly locomotive service charges; that it should have a capacity to lift the long and heavy plate girders used by the railroad companies; that it may be quickly set up in working order and rapidly dismantled for shipment to another job; that it have ample coal and water capacity arranged for effective counterweight; that it should have quickly adjustable devices for blocking and anchoring the car against overturning; lastly, that it have auxiliary devices for controlling main booms and side booms. Special derrick cars, of which some have been protected by patents, have been built by bridge companies, construction companies, and railroad companies. The Mitchell car is the pioneer of the elaborate steel derrick cars. Sketches are given on patent drawing No. 817 862. This car has a central compartment into which the full-length boom slides on the floor of the car and under the engine at the rear, raised to clear the boom. Illustrations are shown in Engineering Record, Aug. 25, 1906, page 218, and Dec. 4, 1909, page 629. The Terry and Tench car is patented; the boom folds up like a knife blade by a hinge at the middle. A short illustrated description is given in Engineering Record, June 29, 1907, page 759. The C. M. & St. P. Ry. derrick cars are fully described in Engineering News, June 24, 1909, page 677. The Boston & Maine R. R. car is described in Engineering Record, Feb. 27, 1909, page 239; this is also the car of the Boston

Bridge Works. Special steel cars have been designed by The Pennsylvania Steel Company, Bridge and Construction Department, and The Phœnix Bridge Company. In Engineering Record of April 17, 1909, page 517, seven types of derrick cars are shown in the illustrations.

It is readily seen that the derrick car has an advantage over a stationary derrick set up at the abutment of a bridge. The

Fig. 23. — Plate Girder Erection. Complete span is placed by derrick car and auxiliary gallows frame. No falsework is used, and traffic is maintained. Lehigh & New England R. R. bridge over Lehigh river at Slatington, Pa.

girders and other members can be picked up near the site and moved out to place; thus there is saved the unloading or double handling, if picked directly from the railroad cars. Plate girder spans are quickly erected, especially the deck type. The single girders and the cross frames are placed with a derrick car, while the lateral bracing is put in by hand; a span riveted up complete is placed in one operation, if the weight is not too great (see Fig. 23). Through plate girder spans furnish more lifts as panel by panel the floor beams and stringers are rapidly connected up. Sometimes the floor system is placed first, on false-

work, when traffic is to be maintained; then the girders are placed and bolted to the floor beam as a last operation, after which the falsework can be released. Figure 23 shows a combination of derrick car and gallows frame erection.

Riveted truss spans have supplanted pin-connected truss spans for short lengths. Falsework is necessary to support them until the last truss member is placed; this is true whether traffic must be maintained or not. The shoes and first bottom chord section are placed at one end and the floor filled in; the end post and web members are bolted to the bottom chord gusset plates, or connection plates; and then the top chord, portal strut, and top lateral bracing complete this braced section. The second bottom chord section with truss follows, and other sections until the span is erected. If traffic is to be maintained, it is desirable to erect the complete floor and bottom chord, properly supported for camber, and then the complete overhead work. This use of the new metal floor eliminates the substantial metal or timber falsework stringers which would otherwise be required to carry traffic.

Pin-connected trusses are used almost entirely for long span bridges, and in those cases are erected most economically with a traveler. Compared with a riveted span, there are many more separate pieces, such as the pin and numerous eyebars at each connection point, which must be held temporarily while the boom lifts the main member or top chord section. The entire floor and bottom chord eyebars are placed first, and when the posts have been bolted to the floor beams, the bottom chord pins can be driven while the boom holds the diagonal eyebars. The end post and top chord are placed, the top pins driven, and the top lateral system closed up. When the spans are short, the erection is so rapid that the derrick car picks the material from the cars on which shipment was made; while for long spans the

derrick car can replace the yard derrick, unload and distribute material over a large space to better advantage for erection.

The derrick car is best suited for viaduct erection, next to plate girder erection; it is particularly efficient in unloading and sorting the large number of pieces required for main members and bracing. Beginning at the abutment, it erects the first bent or braced tower and places the girder span. If the main span is too long for the boom to reach across, a splice section is added to the boom, or a gaff attached to the forward end. The car then moves forward to erect units of tower, main span, and tower span. If the tower is high or main span unusually long, the bottom section of the braced tower is erected. Then the near bent is completed to the top, so that the main span can be placed and the car move ahead so that the tower may next be completed and the tower span placed.

The development of the locomotive crane, now being used to an increasing extent as an erection tool, results from its use as a portable derrick in industrial plants. Its advantage lies in the fact that it can lift full capacity for the entire circular area within reach of its boom; on the other hand, the derrick car must be blocked or anchored for lifting 10 to 20 degrees to the side, thus becoming stationary. The locomotive crane is practically a light capacity, light weight derrick car and can be used at a distance from regular railroad tracks. On light building work it is able to do very rapid work, due to the light weights and its speed when making lifts. A record in erection was made on the Chelsea Piers in New York City, as noted with illustrations in Engineering News, Jan. 14, 1909, page 29, when two locomotive cranes were used, one on each side of the pier, erecting columns and girders and both together raising the roof trusses.

Frequently weights of girders are beyond the capacity of derrick cars, or the erectors do not have the latter. In those

cases the railroad wrecking cranes, capable of lifting 50 to 100 tons, are pressed into service; but they have the disadvantage of being taken away at any moment for use at wrecks. The

Fig. 24. — Viaduct Erection. Shows steel derrick car with 107-foot boom. Tower and spans are used temporarily to support pin span. Carolina, Clinchfield & Ohio R. R. bridge over Broad river.

short booms limit their reach, so that two cranes are used, one at each end, when lifting the long heavy girders. If only one crane is available, the additional lift is made by a derrick car, derrick, or gallows frame. Another valuable erection appliance

is the lighter, or floating, derrick, used in the harbors of large cities and capable of being floated to a seacoast inlet. These floating derricks can pick loads from 75 to 200 tons; their long booms provide a reach that commands an entire bridge site. They are independent of the condition of the approaches and need not wait for tracks because material can be floated to the place required. For plate girder work, especially when there are many adjacent spans, they secure low cost erection.

ART. 27. FALSEWORK.

Long and heavy spans require elaborate temporary falsework and plant equipment for their erection. Supporting falsework must be used under the structure; movable framework, called the traveler, overhead. The development of the traveler and its use in rapid bridge erection is due to the American pin-connected truss span. The original solid front of falsework has given way to a framework with openings to secure the least obstruction to passage through or underneath. Falsework will be treated under three heads: falsework conditions with and without traffic to maintain, types of falsework, and falsework in deep water. The travelers naturally group themselves under three heads: gantry travelers, projecting travelers, and tower travelers for mill buildings. Travelers and falsework in steel will be considered finally.

Falsework is used to carry the weight of the new metal structure until it is made self-supporting by completing the last truss connection. On top of the falsework a runway of stringers is provided to carry the weight of the moving traveler to the bents, close to which the traveler legs must stand when lifting is being done. Falsework can therefore be comparatively light when there is no traffic to maintain, because the load is static. Modern heavy train loads require heavy falsework to

maintain the traffic during erection; they, however, move across the span at greatly reduced speed. The expense of the falsework is chargeable to the new span and should therefore be kept as low as possible. For deck spans, an extra story of falsework, called top falsework, must be added when traffic is maintained. It is built between the trusses and extends from the top chord, where it carries the track, to the main falsework underneath the bottom chord.

The falsework unit is the bent. It consists of two horizontal pieces, two inclined legs, and a varying number of vertical legs dependent on width of bridge and traffic conditions. The top horizontal piece is called the cap, the bottom is called the sill, and the minimum number of legs is four. The bent is thoroughly braced by planks, one on each side, running in opposite directions between diagonally opposite corners, this bracing being called X-bracing. The bents are spaced so as to form panels of equal length between the abutments or piers; there is a bent close to the masonry at each end to serve as a stringer support. On top of the falsework, stringers extend over the full length of the span and are bolted to the caps of the falsework. These stringers are placed underneath the new or old truss to serve as support for blocking or jacks; the others serve to carry the material track and also the traveler track. A second line of longitudinal struts extends full length of the span, butting against the masonry at each end close to the water surface. Two bents are braced longitudinally with X-bracing to form a braced tower with one or more panels between, in which the bracing has been omitted. When spans are long and high above water, the bents must be made wide at the base to give stability against lateral overturning. The bents are built in stories and braced as such, when the height requires, with a line of longitudinal struts at each story. The amount of material

finally becomes excessive, as can be seen at the Rondout Viaduct shown in Engineering Record, April 7, 1906, page 440. A falsework bent is designed for a full panel load plus 50 percent excess, 25 percent from each adjacent panel, due to unequal settlement or from the jacking when it is resorted to. The compression at each joint is found to be $\frac{1}{4}$ inch, due to irregular sawing and to end fibres of legs crushing into cross fibres of cap or sill. The load that can be applied on each leg is therefore limited by the crushing strength per square inch, across the grain of the wood of the cap or sill. Loads are sometimes heavy enough to require double bents.

Falsework in deep water is a difficult and dangerous proposition with which to deal. Piles must be generally driven to secure proper foundation for the falsework bents. To estimate the number of piles required, a frictional resistance per square foot of pile surface is assumed at 100 pounds for alluvial or semifluid bottom, 200 pounds for compact silt, and 300 to 500 pounds for sand to sand-gravel bottom. The piles for erection use are loaded from 10 to 40 tons, and the penetration should be 40 feet in semifluid to 10 feet in good sand bottom. It is advisable to have a double thick cap on the pile bents to properly distribute the load. Pile bents are usually braced transversely above the water surface only; failure has resulted from the long unbraced section in deep water or from the bottom's being scoured away and producing this condition. Even when falsework is strong enough for the superimposed load, there is danger of drift lodging against it and causing a wreck in time of high water by scouring the bottom or lifting or floating the falsework. In order to eliminate this danger it is becoming customary to use one or more temporary girder or truss spans, just above the water surface or close to the metal work, seated on temporary falsework towers.

ART. 28. TRAVELERS.

The most familar type of traveler is the gantry traveler. It consists of two framed bents straddling the structure, braced together longitudinally. Since for long spans the traveler must be high, it is therefore built of three bents thoroughly braced, with a platform at the base for carrying hoisting engines. For smaller spans, the engine is kept on shore, — the lead lines to the tackles are supported at intervals on projecting timber arms on the span. The traveler is equipped with a full set of tackle to do all necessary lifting for the erection of the given span. In the case of two bent travelers, for single and double track spans, the loads lifted range from 7 to 12 tons, the weight of the traveler ranges from 35 000 to 80 000 pounds and contains from 8500 to 19 500 B. M of lumber. Photographs of early erection work show a scaffold, or framework, for the full length of the span, resting on the falsework. The truss was erected from this framework; the tackle was shifted from point to point as it was required. To reduce this time and cost, the traveler was evolved, and thus the framework and tackle might be quickly shifted to the point where needed. With the movable traveler the work of erection begins at the fixed end of the span. The shoes are placed on the intersection of the pin and truss center lines furnished by the railroad company's engineer. The chords are lined out or assembled; the traveler moves along and places the members until the expansion end is reached; the traveler is returned to midspan and erects the middle panel or panels of each truss with their transverse and top lateral bracing to complete a braced section. The traveler is moved toward the fixed end to complete that part of the bridge and, returning to the expansion end, completes the bridge. Whenever the traveler is moved, it must be wedged up under its legs to transfer the loads, when lifting, directly to the falsework instead of

through the sill and traveler wheels. The trusses of pin-connected bridges are so designed that the bottom chord is on the circumference of a large circle instead of a level line. This camber must then be provided for in the falsework by means of blocking at the panel points of the truss; the highest at mid-stream, dropping off gradually to nothing at the ends. The fore-

Fig. 25. — Pin Span Erection. Shows typical traveler and falsework. Middle section is erected. Pennsylvania R. R. bridge over Susquehanna river at Havre de Grace, Md.

man guesses the amount of settlement of the foundation or piles and provides for the compression in the falsework. If his assumption is correct, the last connection can be easily closed with the pin; otherwise the truss must be raised or lowered with jacks until the chords are in the proper arc.

When bridges are erected by the cantilever method without supporting falsework underneath, a projecting or overhanging traveler is used. This type varies for each structure, while the gantry traveler is standard in two-bent and three-bent travelers.

The simplest type is a gantry frame straddling the track on which material on cars is moved forward; on top of this frame is placed one or two derrick booms, and at the rear is mounted the engine. For viaduct erection the overhang traveler has been most frequently used. It consists of an arm projecting forward from the gantry frame from which tackle is hung for lifting, or it is equipped with a trolley runway to move the load instead of fleeting it forward with the tackle. This arm is often extended in the rear to counterbalance the forward arm. By referring to an article on the Different Methods of erecting Steel Viaducts, in Engineering Record, April 2, 1910, page 429, the many illustrations will show the variations in this type of erection traveler. Another type of traveler has also been developed to move along the curved top chord eyebars of a cantilever arm. The same principle is used in erecting truss bascule bridges in an open, or raised, position. In the latter case, it climbs up the back or top chord, by panels, until the top or end panel has been erected. The ordinary derrick cannot reach the hundred, or more, feet necessary for lifting the end pieces. This traveler is simply derrick equipment attached to a sloping triangular base and is called an overhead traveler. In the erection of the towers of the Manhattan Bridge in New York City, the derricks with their supporting base climbed up the tower vertically.

The tower traveler, based on the necessity for having a derrick which is freely movable, is elevated above the ground to make high lifts, and is able to cover a wide space with the use of several booms. The tower is well braced, one to three stories high, mounted on wheels to run on a trackway; it has two forward booms at the top and frequently two booms in the rear at the bottom floor elevation. The engine and tools are carried on this lower floor. Its use is most advantageous in the case of long or rectangular buildings known as mill buildings. The

traveler, moving along to place the columns, wall girders, struts, roof trusses, top bracing, and purlins in this order, panel by panel, completes the frame as it goes. This is done rapidly and economically, especially when an even ground surface necessitates little blocking for the runway. Very heavy work has been done by this method in the erection of buildings for an open hearth steel plant.

Steel falsework travelers and erection tools become more necessary as long heavy timber becomes scarce and weights of structures increase. Gin poles and derricks, standardized as to size and capacity, are built of structural steel. Steel wire rope, instead of manilla rope, is used for operating, while the motive power is rapidly changing to the electric hoist. Even the old style hand crab is built of steel for heavy work. Gallows frames and traveler bents now have their top trusses or caps made of steel adjustable to varying conditions. Special travelers for viaduct erection are usually built of steel. Rolled beams are used for the deck of falsework which is also made of steel for very heavy work. In the case of the Cornwall Bridge, Ontario, Canada, steel falsework had to be used because it was impossible to place and hold down timber legs in the deep, swift current. When permanent steel work, such as plate girder approach spans, are used as a temporary support over waterways and roadways, much falsework is saved, and the risk from floating driftwood is lessened. In viaducts it is possible to use parts of towers and girder spans to support trusses spanning the deep part of the ravine section. In other words, timber and the expense of framing are saved wherever possible by using new steel or borrowed permanent steel for temporary support.

CHAPTER V.

TABLES AND STANDARDS.

ART. 29. MANUFACTURERS' HANDBOOKS.

The student of bridge design will find it absolutely necessary to have at hand one of the handbooks issued by the manufacturers of structural materials. There are a number of these, the best known being those popularly called the Phœnix, Carnegie, Pencoyd, Jones and Laughlins, Passaic, and the Cambria handbooks. The titles of these books are as follows: Useful Information for Architects, Engineers, etc., issued by the Phœnix Iron Works, Phœnixville, Pa.; Pocket Companion, issued by the Carnegie Steel Company, Pittsburg, Pa.; Steel in Construction, edited by James Christie, and issued by A. and P. Roberts Company, Philadelphia, and the American Bridge Company, New York; Standard Steel Construction, issued by Jones and Laughlins, Limited, Pittsburg, Pa.; Structural Steel and Iron, edited by G. H. Blakeley, and issued by the Passaic Rolling Mill Company, Paterson, N. J.; Cambria Steel, issued by the Cambria Steel Company, Johnstown, Pa., and Structural Steel, edited by G. H. Blakeley, and issued by the Bethlehem Steel Company, South Bethlehem, Pa. The Pencoyd, Cambria, Bethlehem, and Carnegie handbooks are the most complete and in general use for work in bridge design. The Passaic and the Jones and Laughlins handbooks are mainly adapted for the design of steel buildings, but are also used in bridge design.

The handbooks contain full tables of all the market shapes of steel manufactured by the respective firms, stating weights, areas of sections, positions of centers of gravity, moments of

inertia, radii of gyration, and other constants. Tables are also given for the shearing and bearing values of rivets, the bearing values of pin plates, the resisting moments of pins, standard bolts, eye-bars, bridge pins, and other details, as well as for the weight and strength of materials used in bridge and building construction. The necessary computations in bridge design may be greatly shortened by the use of these tables. In the following pages a few tables are presented which are more complete than those in the handbooks.

In order to obtain uniformity in the work done at its various plants, the American Bridge Company issued, in 1901, a book entitled Standards for Structural Details. It contains a number of tables similar to those in the handbooks as well as some new ones, together with details relating to the use of corrugated steel for roofing and siding, and of standard doors and windows. The rules for making shop drawings are referred to in Art. 17, and some of them are reprinted.

OSBORN's Tables of Moments of Inertia and Squares of the Radii of Gyration economize time in designing the struts in lateral and sway bracing, and the posts and upper chords of trusses. BUCHANAN's, SMOLEY's, or HALL's Tables of Squares are useful in finding the lengths of diagonal members.

ART. 30. GENERAL SPECIFICATIONS.

A number of general specifications for steel railroad bridges and viaducts have been prepared by consulting engineers, and are published for general use, including those of COOPER, WADDELL, THE OSBORN COMPANY, THACHER, BOUSCAREN, SEAMAN, and SCHAUB. COOPER's Specifications have been in use for many years, and probably more bridges have been built in accordance with them than with any other set. WADDELL'S Specifications are much more elaborate and explicit than the

others, and are particularly serviceable to students, since they embody recommendations based on experience in regard to a considerable number of details whose determination is not wholly subject to theory and to which no reference is usually made.

Many of the leading railroads have their own standard specifications, the most noted being those of the Pennsylvania, New York Central Lines, Baltimore and Ohio, Norfolk and Western, Harriman Lines, Rock Island Lines, Kansas City, Mexico and Orient, and Western Pacific railroads.

The first edition of the General Specifications for Steel Railway Bridges adopted by the American Railway Engineering Association was issued in 1906, and revised in 1910. They are gradually being adopted by railroads throughout the country as their standard.

These specifications usually indicate the types of bridges for different spans, the clearance required for the trains, the character of the wooden floor, the dead, live, wind, and traction loads, allowance for impact and vibration (in some cases), and the safe unit stresses. They also give the general limits in designing, including the minimum thickness of material and sizes of shapes, the general principles in designing structures, the details of riveting, the details of design and construction of beam, plate girder, riveted girder, and pin-connected spans, as well as of viaducts, the quality of material and workmanship, inspection, painting, erection, and final test.

A study of the provisions of the specifications adopted for any given design is a necessary preliminary to the computations and drawing involved in making the design. It will greatly facilitate the student's work to be provided with an index referring to the various paragraphs to be consulted in designing the different parts of the bridge, so that all of them may be duly considered at the proper time.

A comparison of the specifications referred to above will show to the student marked differences in the allowable unit stresses prescribed. One of the principal reasons for this lies in the fact that some make the allowances for impact and vibration by increasing the live load stresses by a percentage, which may be either fixed or variable, while in others these allowances are made by modifying the safe unit stresses. The differences relating to many other details of design and construction will be referred to in connection with the designs in Chapters VII and IX.

General specifications for steel highway bridges have also been prepared by the first four consulting engineers mentioned at the beginning of this article, those of WADDELL and COOPER having been very extensively employed. Many of the manufacturers of bridges also issue such specifications, those of the American Bridge Company being the most complete of that class. Some of the standard specifications for railroad bridges adopted by the railroads also contain provisions relating to highway bridges, as well as to roofs and buildings. FOWLER'S general specifications relate exclusively to steel roofs and buildings.

Most of the specifications for railroad and highway bridges contain an appendix consisting of tables of maximum moments and shears, coefficients of impact, equivalent uniform loads, permissible unit stresses for columns, and other useful data.
The 1902 specifications of the New York Central and Hudson River Railroad contained plates showing standard details for beam, plate girder, and riveted truss bridges. Those relating to plate girders are reproduced in Figs. 43, 44, 46, and 56, and those for riveted bridges are given on Plate VII.

The following references contain important discussions of bridge specifications:

Working Stresses for Railroad Bridges. Editorial. Railroad Gazette, vol. 30, page 797, Nov. 4, 1898.

The Launhardt Formula, and Railroad Bridge Specifications. By HENRY B. SEAMAN. Transactions American Society of Civil Engineers, vol. 41, page 140, June, 1899.

The Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel. By E. HERBERT STONE. Trans. Am. Soc. C. E., vol. 41, page 467, June, 1899.

Proposed Specifications for Steel Railway Bridges. By J. W. SCHAUB. Journal Western Society of Engineers, vol. 5, page 347, Oct., 1900.

Notes on Specifications for Bridge and Structural Steel. By P. S. HILDRETH. Railroad Gazette, vol. 33, p. 517, July 19, 1901.

Highway Bridge Design and Construction. Editorial. Engineering Record, vol. 44, page 217, Sept. 7, 1901.

On Specifications for the Strength of Iron Bridges. By JOSEPH M. WILSON. Trans. Am. Soc. C. E., vol. 15, page 389, June, 1886. Although the specifications given in this paper have been superseded by later ones, the discussion still contains much useful material for the student.

The following articles also contain the results of tests and discussions relating to safe unit stresses:

What is the Life of an Iron Railroad Bridge? By J. E. GREINER. Trans. Am. Soc. C. E., vol. 34, page 294, Oct., 1895.

The Condition of Steel in Bridge Pins. By A. C. CUNNINGHAM, Trans. Am. Soc. C. E., vol. 36, page 91, Dec., 1896.

The following reference gives the revised specifications for structural steel adopted by the American Section of the International Association of Testing Materials: Proposed Standard Specifications for Steel for Bridges, Ships, Forgings, etc. Engineering News, vol. 46, page 11, July 4, 1901.

ART. 31. LIVE LOADS FOR HIGHWAY BRIDGES.

The specifications referred to in Art. 30 give the loads to be used in designing bridge structures which shall have sufficient strength under the various conditions indicated. Such loads, of course, are supposed to closely represent the maximum weights to which the structure is liable.

The extensive use of road rollers, traction engines, electric cars, and other vehicles carrying heavy loads, requires the specification of concentrated loads in designing highway bridges in addition to the uniformly distributed loads, which are supposed to represent the weight of a crowd of people.

According to the specifications of the American Bridge Company, highway bridges are divided into six classes, viz.:

Class A. — For city traffic.

Class B. — For suburban or interurban traffic with heavy electric cars.

Class C. — For country roads with light electric cars or heavy highway traffic.

Class D. — For country roads with ordinary highway traffic.

Class E 1. — For heavy electric street railways only.

Class E 2. — For light electric street railways only.

In designing the floor and its supports, a concentrated live load on two axles, 10 feet between centers, of 5 feet gauge, and assumed to occupy a width of 12 feet, is to be placed on each street car track, this load being 24 tons for classes A and B, and 18 tons for class C; or a concentrated load having the same distribution, width and gauge, is to be placed on any part of the roadway, the load being 24 tons for class A, and 12 tons for classes B and C. Upon the remaining portion of the floor, including sidewalks, there is to be placed a uniformly distributed load of 100 pounds per square foot for classes A, B, and C.

For the floor and its supports of class D, the load shall be either a concentrated load of 6 tons distributed as for the other classes, or 80 pounds per square foot of total floor surface.

In designing the trusses for spans up to 100 feet, the live load per linear foot of car track and assumed to occupy a width of 12 feet, is to be 1800 pounds for classes A and B, and 1200 pounds for class C, while that upon the remaining floor surface is to be 100 pounds per square foot for class A, and 80 pounds per square foot for classes B and C. For spans of 200 feet or more the load per linear foot of track is 1200 pounds for classes A and B, and 1000 pounds for class C, while the load on the remaining floor surface is 80 pounds per square foot for class A, and 60 pounds per square foot for classes B and C. For the trusses of class D the live load is 80 pounds per square foot of total floor surface for spans up to 75 feet, and 55 pounds for spans of 200 feet and over. For intermediate spans the loads are in all cases to be reduced proportionally from the higher to the lower values.

The bridges of class E 1 are designed for those loads which relate to the car tracks only in class A or B, while the bridges of class E 2 are designed for the corresponding loads in class C.

This classification of highway bridges and the corresponding live loads are substantially the same as those in COOPER's specifications, which were published a few months before.

The loads specified by WADDELL differ considerably from the above. The concentrated loads are distributed to three wheels in the case of the road roller, and to four wheels in the other cases. The weights and spacing of wheels are given for several classes of electric cars, together with the equivalent uniform loads. The uniformly distributed load is determined by means of a diagram, the length of the load on the span being considered, which causes the maximum stress in any given member of the truss or floor. In the case of bridges with exterior

sidewalks it is also stated what portion of the roadway and sidewalks shall be loaded for finding the stresses in the trusses and the floor beams.

ART. 32. LIVE LOADS FOR RAILROAD BRIDGES.

The General Specifications for Steel Railway Bridges of the American Railway Engineering Association, which are gradually replacing the specifications formerly adopted by individual railroad systems, give a live load, for each track, of two typical consolidation locomotives followed by a uniform load, according to COOPER's series. The loads and spacing for COOPER's class

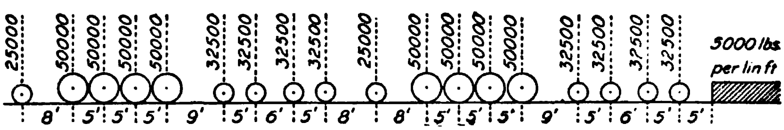


Fig. 26.

E 50 are shown in Fig. 26. The spacing is the same for all classes, and the loads on the axles, expressed in pounds, are arranged so that the corresponding loads of any two classes have the same ratio as their class numbers. For example, the loads of class E 60 are six-fifths of those of class E 50. Any stresses, bending moments, or shears, due to these loadings are therefore proportional to their class numbers.

The same specifications give alternative loads on two axles, seven feet apart, to represent the heavier drivers of passenger locomotives, and are to be used when they cause greater stresses than the loads on the driver axles of the typical consolidation locomotive. This will occur in beam bridges of very short span and in the floor system and sub-verticals of truss bridges whose panels are unusually short. Each of the alternative loads is 25 percent heavier than the corresponding axle load on drivers. In COOPER's specifications of 1906, however, the alternative loads are given as 50 000 pounds on each of the two axles, six feet

apart, for class E 40 or less, and 60 000 pounds each for all classes above E 40.

WADDELL's compromise standard, class Q, is shown in Fig. 27. The other classes are designated as R, S, T, etc., the spacing remaining the same. The axle loads for each class are derived from those of the preceding class by subtracting the constant

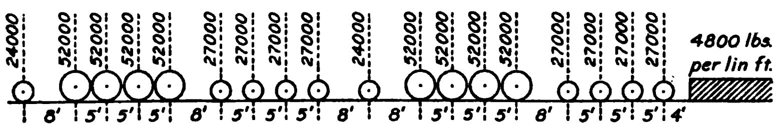


Fig. 27.

difference of 1000 pounds for the pilot axle and each tender axle, 3000 pounds for each driving axle, and 200 pounds for the uniform load per linear foot. The alternative loads for class Q are 58 000 pounds each on two axles only, seven feet apart, and 2000 pounds are subtracted from each load for each succeeding class in the series.

The locomotive loads adopted for designing bridges in the United States from 1886 to 1903, together with an estimate of what increase, if any, may be expected in the future, were given in a paper on 'Live Loads for Railroad Bridges,' presented at the International Engineering Congress at St. Louis, in October, 1904, by HENRY W. HODGE, and published in the Transactions of the American Society of Civil Engineers, 1905, vol. 54, part A, page 79. The paper is followed by discussions on pages 87 to 109, which include diagrams of bending moments, shears, and equivalent uniform loads, for a number of different loadings, and for spans up to 200 feet. See also various references relating to this subject given in Part I, Art. 40.

With the aid of such diagrams of moments and shears, each type of locomotive in use on a railroad system may be evaluated in terms of COOPER's series, and the live load capacity of every bridge may be expressed in similar terms.

ART. 33. RIVET PROPORTIONS.

The rivet proportions given in the following table are the Pencoyd standard adopted by the American Bridge Company. For finished or button heads the diameter equals $1\frac{1}{2}$ times the diameter of the shank plus one-eighth of an inch, and the height equals 0.425 times the diameter of the head. For countersunk

SIZE.	FINISHED HEAD			COUNTERSUNK.	
Diameter.	Height.	Diameter.	Radius.	Depth.	Diameter
$\frac{3}{8}$	$\frac{19}{64}$	$\frac{11}{16}$	$\frac{7}{16}$	$\frac{3}{16}$	$\frac{19}{32}$
$\frac{1}{2}$	$\frac{3}{8}$	$\frac{7}{8}$	$\frac{9}{16}$	$\frac{1}{4}$	$\frac{25}{32}$
$\frac{5}{8}$	$\frac{29}{64}$	$1\frac{1}{16}$	$\frac{43}{64}$	$\frac{5}{16}$	1
$\frac{3}{4}$	$\frac{17}{32}$	$1\frac{1}{4}$	$\frac{51}{64}$	$\frac{3}{8}$	$1\frac{3}{16}$
$\frac{7}{8}$	$\frac{39}{64}$	$1\frac{7}{16}$	$\frac{59}{64}$	$\frac{7}{16}$	$1\frac{3}{8}$
1	$\frac{11}{16}$	$1\frac{5}{8}$	$1\frac{1}{32}$	$\frac{1}{2}$	$1\frac{9}{16}$

(All dimensions are in inches.)

rivets the depth of head equals one-half of the diameter of the shank, the bevel of the head being 60 degrees. Since the finished head of a rivet is not quite a hemisphere, the diameter of its base is a little less than twice the radius of its spherical surface.

According to the Cambria standard, the height of the finished head equals six-tenths of the diameter of the shank, while the radius of the head equals three-fourths of the diameter of the shank plus one-sixteenth of an inch. The diameter of the countersunk head is made the same as that of the button head. This standard also recommends that in figuring clearances for rivet heads, a height of $\frac{5}{8}$ inch should be allowed for $\frac{3}{4}$-inch rivets, and of $\frac{3}{4}$ inch for $\frac{7}{8}$-inch rivets.

The clearance between any rivet head and an adjacent surface or projection must allow room for the riveting tool, — at least $\frac{3}{8}$-inch clearance is required. For a $\frac{7}{8}$-inch rivet the distance from

the center of the rivet to the back of an adjacent angle should not be less than $1\frac{1}{4}$ inches, while for a $\frac{3}{4}$-inch rivet it should not be less than $1\frac{1}{8}$ inches. The minimum distance between the center of a rivet in one leg of an angle and the projection of a rivet head located on the other leg is $1\frac{1}{8}$ inches. This clearance determines the minimum stagger.

When stiffeners are crimped over the flange angles, the distance from the edge of the flange angle to the next rivet in the stiffener should be $1\frac{1}{2}$ inches plus twice the thickness of the flange angle, but never less than 2 inches. The distance from the center of a rivet to the edge of a plate should not be less than $1\frac{1}{2}$ diameters; and whenever practicable, it should be at least 2 diameters, and it should not exceed 8 times the thickness of the plate.

The maximum pitch of rivets in the direction of the stress should not exceed 6 inches nor 16 times the thickness of the thinnest outside plate, and the minimum pitch should not be less than 3 diameters of the rivet. Additional requirements relating to the pitch and location of rivets will be given in connection with the designs in the following chapters. Rivets should not be countersunk in plates whose thickness is less than the depth of the countersunk head, and it is preferable that the plate should have some bearing in the shank of the rivet.

The rivets chiefly used in bridge work are $\frac{7}{8}$ inch in diameter, but $\frac{3}{4}$-inch and $\frac{5}{8}$-inch rivets are employed in lacing or in other minor details. In some very heavy work rivets 1 inch in diameter are being used. Rivet tests show that the grip length should not exceed 5 diameters for machine-driven rivets. See Engineering News, vol. 24, page 500, December 6, 1890.

ART. 34. RIVET SPACING IN ANGLES.

The following table gives the standard spacing adopted by the American Bridge Company, together with the maximum

diameters of the rivets to be used. As shown in Fig. 28, l represents the length of the angle leg, a the distance from the corner to the pitch line of a single row of rivets, this distance being known as the gage, b the corresponding distance to the first row for double riveting, and c the distance between the pitch lines of the two rows. Two values of b and c are given for the 6-inch angle, the first being used when the thickness of the angle does not exceed $\frac{3}{4}$ inch.

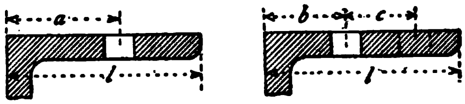


Fig. 28.

LENGTH OF ANGLE LEG l	SPACING a	SPACING b	SPACING c	MAXIMUM DIAMETER OF RIVET.
8	$4\frac{1}{2}$	3	3	$\frac{7}{8}$
7	4	$2\frac{1}{2}$	3	$\frac{7}{8}$
6	$3\frac{1}{2}$	$\{2\frac{1}{4}$, $2\frac{1}{2}$	$\{2\frac{1}{2}$, $2\frac{1}{4}$	$\frac{7}{8}$
5	3	2	$1\frac{3}{4}$	$\frac{7}{8}$
4	$2\frac{1}{4}$			$\frac{7}{8}$
$3\frac{1}{2}$	2			$\frac{7}{8}$
3	$1\frac{3}{4}$			$\frac{7}{8}$
$2\frac{3}{4}$	$1\frac{5}{8}$			$\frac{3}{4}$
$2\frac{1}{2}$	$1\frac{3}{8}$			$\frac{5}{8}$
$2\frac{1}{4}$	$1\frac{1}{4}$			$\frac{5}{8}$
2	$1\frac{1}{8}$			$\frac{1}{2}$

(All dimensions are in inches.)

In several references to this article in succeeding chapters the statement is made that the $3\frac{1}{2}$-inch angles are the smallest in which $\frac{7}{8}$-inch rivets may be used. The designs given in those chapters were made before the revised standards were received, and are in accordance with the former Pencoyd standard in this respect.

ART. 35. PIN PLATE AND RIVET DIAGRAM.

The diagram in Fig. 29 is constructed for the unit stresses and diameters of rivets there given. The diameters of the pins are laid off as abscissas, and the bearing values for the pins as ordinates, the linear bearing on the pins being marked on the lines radiating from the lower left-hand corner. The allowable stress for an 8-inch pin with a bearing of $1\frac{7}{8}$ inches is seen to be 180 000 pounds by following the ordinate for a diameter of 8 inches until it meets the radial line marked $1\frac{7}{8}$ inches, and reading off its value from the scale at the right.

The number of $\frac{7}{8}$-inch rivets in single shear is laid off at the top of the diagram, so that by following down any ordinate until the diagonal line (separating the two systems of horizontal and vertical ruling) is reached, the allowable shearing stress of the corresponding number of rivets may be read off by the scale on the right. Thus, the shearing value of 22 rivets is found to be 99 000 pounds. It may be added that the diagram as here printed is considerably reduced from the original size, on which more precise readings could be made. Usually, however, it is not necessary to read closer than 1000 pounds. On the left side the number of $\frac{7}{8}$-inch rivets in bearing is laid off to such a scale that by following any horizontal line until it intersects a line radiating from the upper left-hand corner on which the thickness of plates, or the linear bearing of the rivets, is marked, the equivalent number of rivets in shear may be read off on the scale at the top. For instance, the bearing stress of 12 rivets in a $\frac{1}{2}$-inch plate is very nearly equal to the stress of 14 rivets in single shear.

By combining the two preceding operations the value of the bearing stress of 12 rivets in a $\frac{1}{2}$-inch plate may be obtained by following down from the point of intersection to the diagonal line, and then reading the stress on the scale at the right, the

PIN-PLATE AND RIVET DIAGRAM.

Unit stresses:
Bearing of pin, 12 000 lbs. per square inch.
Bearing of rivets, 12 000 lbs. per square inch.
Shear of rivets, 7500 lbs. per square inch.

$\frac{7}{8}$-inch rivets:
Single shear, 4510 lbs.
$\frac{3}{8}$-inch bearing, 3940 lbs.
$\frac{7}{16}$-inch bearing, 4590 lbs

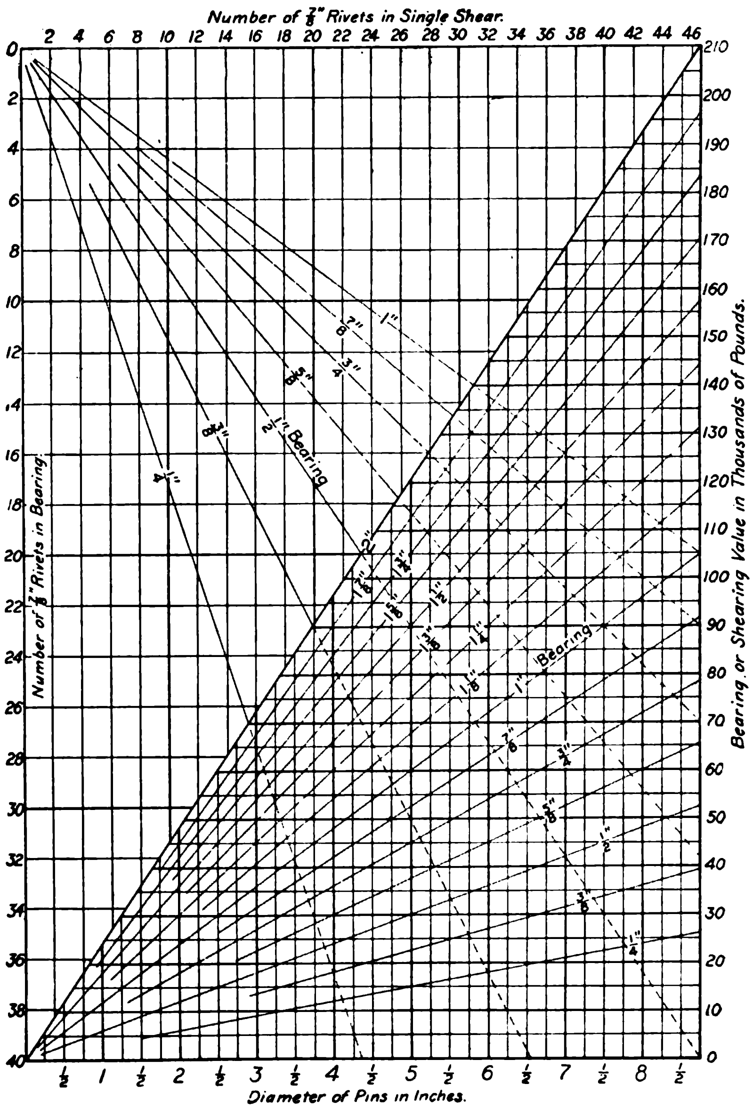


Fig. 29.

value being 63 000 pounds. Furthermore, all of the preceding operations may be combined. For example, the bearing value of a 6-inch pin with a bearing of $1\frac{1}{4}$ inches equals the shearing value of 20 rivets or the bearing value of 23 rivets in a $\frac{5}{8}$-inch plate. The upper radiating lines go beyond the diagonal in order to extend the limits of the diagram.

ART. 36. CONVENTIONAL SIGNS ON DRAWINGS.

Full lines show that they are visible, while invisible lines are represented by a series of dashes of equal length. In order to distinguish between invisible lines of the structure or object and the projecting lines it is desirable to use dashes about $\frac{1}{8}$ inch long for the former and about one-third as long for the latter. The appearance of a drawing is materially improved by making the spaces between the dashes uniform. In general these spaces should measure about $\frac{1}{32}$ inch or a little less than the smaller dashes. If the spaces are longer than the dashes, a line loses its apparent continuity if it is placed close to other lines of a similar character.

Feet are indicated by a prime ($'$), and inches by seconds ($''$), and these are usually placed on dimension lines having arrow points at the ends. These lines should be of two kinds: first, those marking the points, lines, or sections between which the measurement is to be recorded; and second, those drawn at right angles to the preceding lines, with an arrow at each end and the dimension marked at the middle. The former should have the same form as projecting lines, while the latter may either be the same or may be distinguished from both projecting lines and invisible lines of the structure by using very short dashes or elongated dots and spaces nearly or quite $\frac{1}{8}$ inch long. In constructing these lines the pen should be opened about twice as far as for the ordinary lines constituting the greater part of the drawing.

Center lines of plans, elevations or sections, or lines marking the position of sections whose forms are shown elsewhere, are appropriately indicated by the usual convention for traces of auxiliary planes, consisting of very long dashes, say $\frac{3}{8}$ inch, with two dots between them. Center lines of members or rivet lines are indicated either by very light full lines in black or by red lines of ordinary weight. The red lines on tracing cloth usually give faint lines on the blue print which may be readily seen. The sizes of dashes and spaces given above are those suitable for bridge drawings whose scale ranges from $\frac{1}{2}$ to 1 inch to the foot, and should be modified accordingly for scales beyond these limits.

When drawings are to be shaded by making some of the lines heavier than others, the following simple rule decides which lines are thus to be distinguished, plans being treated the same as if they were elevations: If a line separates two surfaces and there is an offset perpendicular to, and toward, the plane of projection in passing from the left-hand or upper surface to the right-hand or lower surface, the line (marking the offset) should be shaded. If the offset is in the opposite direction, that is, if the former surface is nearer the plane of projection (or farther from the observer) than the latter, the line is not to be shaded. When the offset is not perpendicular, as in the case of a beveled or chamfered edge, the form is usually indicated by the presence of diagonals or curves at the ends of the chamfer. If the line marks a rounded edge, its weight should be increased but slightly. Shading adds very materially to the realistic effect of a drawing and enables workmen not trained to the use of drawings to interpret them more readily. On account of the extra labor involved, shading is frequently omitted on shop drawings.

Clearness in detail drawings often demands that cylindrical, conical, spherical, or other curved surfaces should be covered

with shade lines spaced in accordance with the principles of shades and shadows in descriptive geometry.

Cross-sections are usually ruled with parallel lines, drawn light and full, as shown in Fig. 53, Art. 44. Sometimes a standard of section lining is adopted for different materials such as those shown in Fig. 12, Art. 17. The method frequently adopted where it is necessary to make the distinction is to mark those parts composed of any material other than that which constitutes the bulk of the structure by placing the name of the material either on or adjacent to them. When the section is so small that ruling will not appear to be suitable, the section is filled up solid. In order that adjacent sections so represented may appear distinct in form, a small space is left between them, although the shapes are really in perfect contact.

In order to give proper directions for the riveting, conventional signs are employed on the drawing. Two systems are in general use, one being known as OSBORN's code and published in OSBORN's Tables of Moments of Inertia as well as in nearly all of the handbooks, while the other is the Pencoyd system, which is given in the Pencoyd handbook and in the American Bridge Company's Standards for Structural Details. The former system of conventional signs with slight modifications was adopted as recommended practice by the American Railway Engineering Association in 1910.

Where the sign of a rivet head is surrounded by a broken circle of larger diameter, it represents the insertion of a washer to maintain uniform spacing between two angles acting together as a strut or tie. When the terms angles, channels, I-beams, etc., are not marked on the drawing, symbols are used having the form of the section. For additional information regarding shop drawings, see Art. 17 and Chap. X.

CHAPTER VI.

DETAILS OF PLATE-GIRDER BRIDGES.

ART. 37. GENERAL ARRANGEMENT.

The simplest form of a plate girder is composed of a vertical web plate to whose top and bottom are riveted horizontal pairs of angles, and to whose ends vertical angles are attached which serve to transmit the load to the supports. As the ratio of the depth of the web to its thickness increases, it becomes necessary to stiffen the web by fastening additional vertical angles at intervals along the span, these being also arranged in pairs on opposite sides of the plate. (See Fig. 30.) As the span increases, two or more web plates must be used and spliced end to end, while for long spans the flanges also require splicing.

For short spans one end of the girder is permitted to slide upon the support, a bearing or base plate being riveted to the bottom of the girder, and a bed plate bolted to the masonry or other support. When the span exceeds about 75 feet, it is desirable to make better provision for the expansion and contraction due to changes in temperature by introducing rollers between the bearing and bed plates, while for spans which are but slightly, if any, longer hinged bolsters are used in order to avoid the unequal pressure upon the rollers due to the deflection of the girder. At the same time the composition of the flanges and of other details is changed so as to provide for the increased stresses caused by greater loads and spans. (See Plate I, Art. 69.)

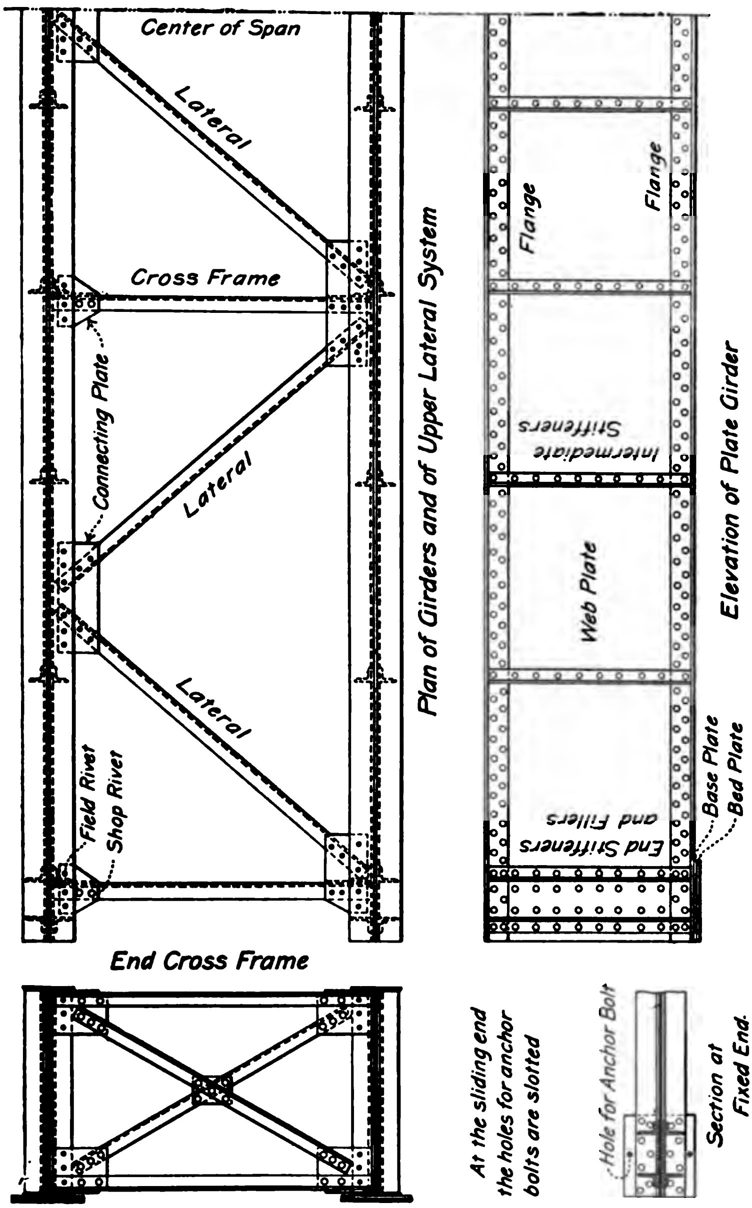

Center of Span
Lateral
Cross Frame
Connecting Plate
Lateral
Lateral
Field Rivet
Shop Rivet
Plan of Girders and of Upper Lateral System
Flange
Flange
Intermediate Stiffeners
Web Plate
End Stiffeners and Fillers
Base Plate
Bed Plate
Elevation of Plate Girder
End Cross Frame
At the sliding end the holes for anchor bolts are slotted
Hole for Anchor Bolt
Section at Fixed End.

Fig. 30.

A plate girder bridge consists of two or more girders connected together by one or two systems of lateral bracing, and by transverse bracing which comprises two or more cross-frames. In a deck bridge the railroad track or highway flooring rests directly upon the tops of the girders, while in a through bridge the floor is attached to the web plates. In one type of the through bridge, transverse girders, or floor beams are connected by gusset plates to the stiffener angles of the main girders to which, in turn, are attached the longitudinal beams or stringers that carry the cross-ties or the planking or floor plates. In another type of construction the transverse beams consist of rolled or built-up shapes that either form a solid or continuous floor, or they are placed so close together as to obviate the use of the stringers. As through plate-girder bridges can have only a lower lateral system, the upper flanges of the girders which are subject to compression must be held in line by means of bracing composed of the floor beams and their angle and gusset plate connections, which are extended up to those flanges for this purpose. When solid floors are employed, a similar arrangement is necessary at corresponding intervals (Art. 45).

ART. 38. THICKNESS OF WEB PLATES.

Experience shows the importance of specifying that the thickness of web plates in railroad bridges shall not be less than three-eighths of an inch, while those in highway bridges and buildings shall not be less than five-sixteenths of an inch. It would be better if three-eighths of an inch were also made the minimum thickness for important highway bridges. In girders carrying heavy loads the magnitude of the vertical shear will frequently require a greater thickness than the minimum value.

Usually the thickness of the web is made the same throughout, as it simplifies the shopwork, the excess of material being offset by the saving in labor. In special cases, however, it may

be necessary to use such a thick web at the ends that it will be economical to vary the thickness either once or twice in the half span. When this is done, filler plates must be placed between the web and the angles on one or on both sides of the flanges, so as to maintain a constant distance between the backs of the flange angles. An illustration of this may be found in the Engineering Record, vol. 43, page 102, Feb. 2, 1901. The web thicknesses of the middle girder are 1″, $\frac{5}{8}$″, and $\frac{3}{8}$″, respectively, and two fillers $\frac{3}{16}$″ thick are used under the flange angles along the $\frac{5}{8}$″ web, while those along the $\frac{3}{8}$″ web are $\frac{5}{16}$″ thick. The outer girders of the same bridge have web plates $\frac{1}{2}$″ and $\frac{3}{8}$″ in thickness, and one filler plate $\frac{1}{8}$″ thick is used under the outer flange angles only.

In the design of plate girders the use of somewhat thicker web plates than is customary should be encouraged because inspectors with extended experience report that they find that the web always gives out first, even in cases where the flanges are thinner. This may be done by specifying that a part of the web section shall be taken as flange area in accordance with the theory of flexure. A thin web gets out of shape more readily in handling, deteriorates more rapidly after it begins to rust, and is more easily injured in case of accident.

Sometimes, instead of changing its thickness, the web plate is reinforced near the ends by riveting a plate on each side of it between the upper and lower flange angles.

ART. 39. COMPOSITION OF FLANGES.

The simplest flange of a plate girder is shown in Fig. 31 and consists of a pair of angles riveted to the web plate with their backs projecting slightly beyond it, so that if the edge of the plate is not perfectly straight, there may be no interference with anything resting upon or attached to the flange.

When additional flange area is required, one or more cover plates are usually riveted to the horizontal legs of the angles as shown in Fig. 32. The number of cover plates should generally be limited so as not to require rivets longer than five times their diameter in order that they may entirely fill the holes when driven. As the stress in the covers must be transmitted indirectly from the web through the angles, it is desirable to use angles whose sectional area shall be equal to or greater than that of the cover plates, and if this is not possible, the largest angles obtainable should be used. Since the bending moment in the girder requires cover plates of different lengths which in turn necessitates notching the cross-ties unequally for deck girders of railroad bridges, the flange has occasionally been

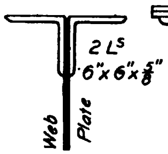


Fig. 31.

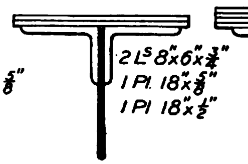


Fig. 32.

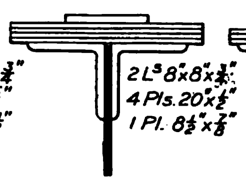


Fig. 33.

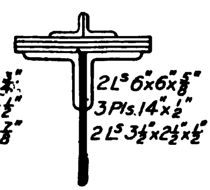


Fig. 34.

modified by making the outer plate narrower than the rest and extending it from end to end of the girder with filler plates placed under it beyond the shorter cover plates. See Fig. 33 and Engineering Record, vol. 44, page 6, July 6, 1901. In Fig. 34 the same object is secured by means of two small angles, the ties being notched over the vertical legs of the angles. The section of the flange at the end of the girder is shown in Fig. 38. With both of these forms of flange the load is brought nearer to the center of the flange. The cross-ties are all alike, and this is a convenience in the removal of ties or in bunching them to secure access below the floor for repairs. The rivets are countersunk so as to avoid interference with the bearing of the cross-ties.

Fig. 35 gives a section used by the Chicago, Milwaukee, and St. Paul Railway in the plate-girder bridges carrying its elevated tracks in Chicago. The top pair of angles extends from end to end, while the lowest pair is the shortest. This flange also secures an even bearing for the track ties without the necessity of boring holes for rivet heads, as is the case generally when cover plates are used, the cross-ties being held in place laterally by notching over the projection of the web plate which extends $\frac{3}{4}''$ or $\frac{7}{8}''$ above the flange angles. The bottom flange has 2 angles $6'' \times 6'' \times \frac{5}{8}''$, and 2 cover plates $14'' \times \frac{5}{8}''$. In another girder of larger span the top angles of the upper flange are increased to $8'' \times 8'' \times \frac{5}{8}''$, thereby securing three rows of rivets to connect the flange angles to the web.

Another method of reducing somewhat the total thickness of cover plates is to insert a vertical plate between each flange angle and the web as illustrated in Fig. 36. This arrangement

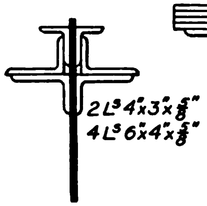


Fig. 35.

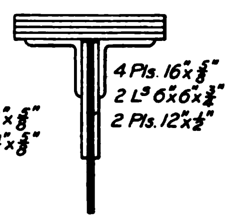


Fig. 36.

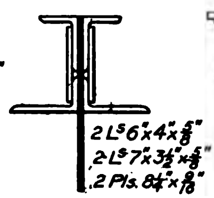


Fig. 37.

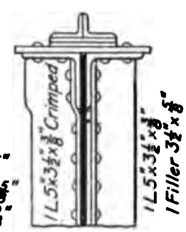


Fig. 38.

has the advantage of permitting three or four rows of rivets to transmit the increments of flange stress from the web to the flanges. The vertical plates are usually made about twice as wide as the angles. Probably the largest flanges of this form which have been constructed are those in the Maiden Lane bridge at Albany, described in Engineering Record, vol. 40, page 474, Oct. 21, 1899. The angles are $8'' \times 8'' \times 1''$, and the vertical plates $16'' \times \frac{5}{8}''$, while the cover plates are $27''$ wide, three being $1''$ thick and the other $\frac{13}{16}''$ thick.

The type of flange which has been used in the heaviest plate girder probably ever built, and in some other girders of very long span, is illustrated in Fig. 41, Art. 41. It consists of 4 angles, with one or more pairs of vertical plates on the faces of the angles, together with a number of cover plates. In the 103-ton plate girder, which is the middle one in a four-track through bridge on the New York Central and Hudson River Railroad over the Clyde River east of Lyons, N. Y., each flange consists of 2 angles 8″ × 8″ × 1″, 2 angles 6″ × 6″ × 1″, 6 side plates $12\frac{1}{2}$″ × $\frac{1}{2}$″, only two of which extend the full length, and 10 cover plates, one of which extends the full length of the girder, 7 plates being 24″ × $\frac{5}{8}$″, and 3 plates 24″ × $\frac{1}{2}$″. The flanges of the outside girders have 2 angles 8″ × 8″ × $\frac{3}{4}$″, 2 angles 6″ × 6″ × $\frac{3}{4}$″, 2 side plates 13″ × $\frac{1}{2}$″, full length, and 5 cover plates, two being 24″ × $\frac{5}{8}$″, and 3 plates 24″ × $\frac{1}{2}$″. This bridge is described and illustrated in the Engineering Record, vol. 43, page 102, Feb. 2, 1901.

Another girder on the Erie Railroad, whose span is 125′ $2\frac{1}{2}$″, has an upper flange of 4 angles 8″ × 6″ × $\frac{5}{8}$″, and 4 cover plates 18″ × $\frac{9}{16}$″, no vertical plates being used. The long legs of the angles are horizontal. See Fig. 41, or Engineering Record, vol. 41, page 565, June 16, 1900.

A modification of this type in which the side plates are included, but the cover plates omitted, is given in SCHAUB'S specifications. The web plate is to project $\frac{1}{2}$ inch above the angles to engage the notches in the railroad track ties. An interesting example of upper flanges without cover plates is given in the Boone Viaduct. See Fig. 37, and Engineering News, vol. 46, page 117, Aug. 22, 1901.

The flanges shown in Figs. 37 and 41 have another advantage in permitting the lateral system to be attached to one of the lower angles, and thus avoiding the trouble from loose rivets in the bracing caused by the deflection of the cross-ties.

Dapping or notching the cross-ties over the cover plates or angles weakens them when the girders are spaced farther apart than the track rails. In those flanges where the cross-tie is notched over the projecting web, or the short vertical legs of bearing angles, as in Fig. 34, the cross-tie takes its bearing outside of the narrow notch, thus preserving its full strength. Less labor is also required in this case, as it is not necessary to bring the top of the notch to an even surface.

ART. 40. WEB STIFFENERS.

As pointed out in Art. 57, the theory of the distribution of stresses in the webs of plate girders and of the functions of intermediate web stiffeners is in an unsatisfactory state, and as a natural result the practice in the use of stiffeners varies considerably. An examination of the drawings of a large number of plate-girder railroad bridges of recent design indicates that for intermediate stiffeners, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles are mostly used for spans below 50 feet, $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles for spans from 50 to 100 feet, and $6'' \times 4'' \times \frac{3}{8}''$ angles for spans above 100 feet. Thicknesses of $\frac{7}{16}''$ or $\frac{1}{2}''$ are employed in very few cases. In general the stiffeners for highway girders and buildings are somewhat lighter.

The end stiffeners usually consist of four angles for ordinary spans, one pair being placed at each end of the shoe or bearing plate, while for long spans two additional pairs of angles are placed midway between them, as shown in Figs. 43, 46, 48, and 49, Arts. 43 and 44.

Fillers are always employed under the end angles, so that the latter may be riveted on straight, as shown on the right of Fig. 38. Intermediate stiffeners are sometimes crimped over the flange angles, as indicated on the left side of the same figure, while in other cases fillers are used. When stiffeners connect

to cross-frames, it is preferable to use fillers. Some engineers prefer to limit crimping to about ½ inch, using fillers only for the excess thickness of the flange angles. When vertical side plates are used in the flanges, fillers of the same thickness are inserted between the upper and lower side plates, while the stiffeners are crimped over the flange angles.

For a novel arrangement of end stiffeners and fillers in which the angles form a continuation of the upper flange angles without reduction of thickness, see Railroad Gazette, vol. 28, page 769, Nov. 6, 1896, and the inset of Engineering News for Aug. 20, 1896. In the former case the intermediate stiffeners, which are about 7 feet apart, do not extend over the upper flange angles.

ART. 41. WEB SPLICES.

The processes of manufacture and the available equipment necessarily impose limitations to the size and weight of web plates, so that large plate girders require a number of web splices. The simplest form of splice shown in Figs. 46 and

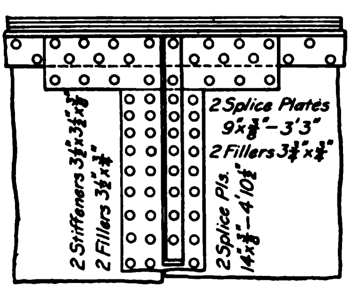


Fig. 39.

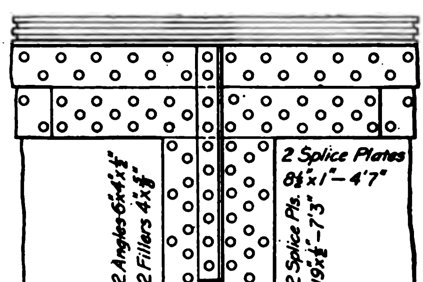


Fig. 40.

49, Art. 44, consists of plates whose length equals the clear distance between the flange angles, and which are riveted to each of the two web plates by two or more rows of rivets. A pair of stiffener angles is generally attached to the splice.

A more efficient splice is that designed in Art. 56 (see Fig. 68). The two flats riveted to the vertical legs of the flange angles not only splice the part of the web not reached directly by the other plates, but add considerably to the strength of the whole splice, since the value of any rivet in resisting the bending moment at the joint is proportional to the square of its distance from the neutral surface. A rivet at the neutral surface can resist shear, but no bending moment. Such a splice is used on the girders of the New Kinzua Viaduct described in Engineering Record, vol. 42, page 510, Dec. 1, 1900. If additional strength be required, the flats over the flange angles

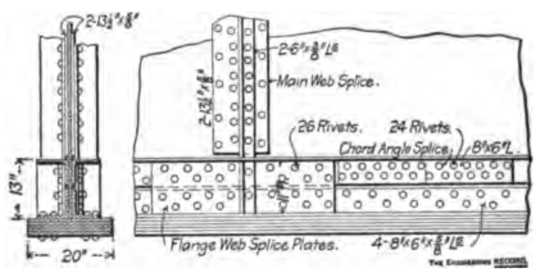


Fig. 41.

may be widened so as to engage one or two extra rows of rivets, filler plates of the same thickness as the flange angles being placed between them and the web. See Fig. 39 and also Engineering News, vol. 30, page 140, Aug. 17, 1893. Fig. 40 shows the web splice in a very heavy girder whose flanges contain side plates.

Longitudinal splice plates for the web are well adapted to the case where the flange has four angles, as in Fig. 37, Art. 39. The splice illustrated in Fig. 41 is the one used on the plate girder of 125′ 2½″ span referred to in Art. 39.

On the Duquesne approach of the Monongahela River bridge at Rankin, Pa., the splices of some girders, whose span is about

118 feet, were made by placing the longitudinal plates alongside of the vertical flange plates, the vertical splice plates being put between these. The use of vertical flange plates requires either the arrangement just mentioned, thereby reducing the relative effectiveness of the splices, or the use of many fillers to move the longitudinal splice plates farther from the neutral surface of the girder. The total thickness of fillers may be reduced by crimping the stiffeners over the longitudinal splice plates.

It is customary to place a pair of stiffeners at each web splice, but in the girders last mentioned most of the splices are located in the spaces between the stiffeners.

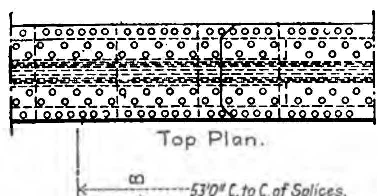
ART. 42. FLANGE SPLICES.

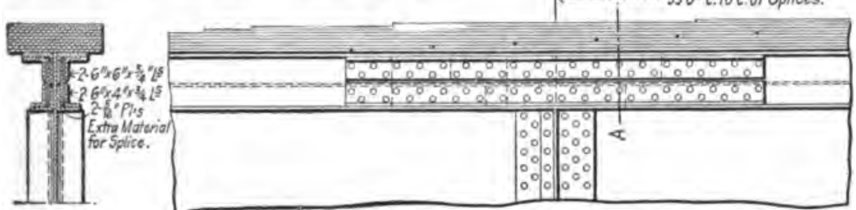
For spans of plate girders less than 60 feet, it is possible to avoid splices in the flanges, as angles and cover plates extending the full length required may be readily obtained. It is frequently economical to pay the additional price that may be asked for extra lengths of such material in order to avoid splices altogether, or to reduce their number to a minimum. No two pieces of either the web or the flange should be spliced within a certain distance of each other, that distance being such as to enable one cut to be fully spliced before the next one is reached.

When each flange consists of only two angles and cover plates, it is customary to splice the outside angle on the left of some web splice, and the inside angle on the right of the web splice in one of the flanges, and to reverse this arrangement in the other flange. The flange angles are usually spliced by means of cover angles whose roots are rounded to fit the fillets of the other angles. The most convenient arrangement for the cover splices is to place them so that in each case the

outer cover near the splice may be extended a sufficient distance to form the splice plate.

Fig. 42 shows the splice of a flange with an exceedingly heavy section. On account of the necessity of shipping this girder in three pieces, the entire flange splice had to be confined to a comparatively short length. The figure shows the location of the splices in the web, in 4 angles,





Section A-B. Top Chord Splice. Center Girder. THE ENGINEERING RECORD.

Fig. 42.

2 side plates, and 6 cover plates, besides the ends of the remaining 4 cover plates, some of which are extended so as to act as splice plates. The composition of this flange of a 103-ton girder was given in Art. 39.

In Fig. 72, Art. 61, and on Plate I, details are shown in which both a cover angle and a flat placed on the opposite side are used to splice one of the flange angles.

ART. 43. LATERAL AND TRANSVERSE BRACING.

The upper and lower flanges of deck plate girders are held in line by means of a series of braces, each one being composed of one or two angles, which together with the flanges form horizontal trusses, known as the upper and lower lateral systems respectively. These systems are most frequently of the Warren type, the panel points of the upper system being

directly above points which are midway between the panel points of the lower system. Transverse or sway bracing is placed at the ends of the girders and at intervals between. These cross-frames usually consist of two horizontal struts and two diagonals, which together with a stiffener on each girder form a rigid rectangular panel. (See Fig. 44.) Similar horizontal struts are often inserted at other points between the cross-frames. The end cross-frames sometimes have diagonals composed of channels instead of angles, while in exceptional cases a solid web plate has been employed. The lower horizontal strut is occasionally omitted in intermediate frames.

In double-track deck bridges the inner girders supporting each track are connected by struts of single or double angles to keep them at a fixed distance apart, no diagonals being employed. These are also shown in Fig. 44. The object of the $\frac{3}{4}''$ fillers inserted between the lateral connecting plates and the flanges is to secure more clearance between the lateral braces and the cross-ties.

Fig. 45 gives a view of two adjacent deck plate-girder bridges on the Baltimore and Ohio Railroad, taken before the track was put in place on one of them. This shows the general character of the upper lateral system and the cross-frames as well as the form of their connections to the girders.

In double-track through bridges both tracks are generally supported between two girders, and the lateral system is then preferably made of the rectangular type, two sets of diagonals being inserted between each pair of floor beams, the latter acting as the struts of the system. The same arrangement is used for single-track through girder bridges, the laterals being riveted to the stringers in both of these cases so as to transfer directly to the girders any stresses due to the braking of trains, or to other longitudinal forces, which might otherwise cause the floor beams to bend horizontally.

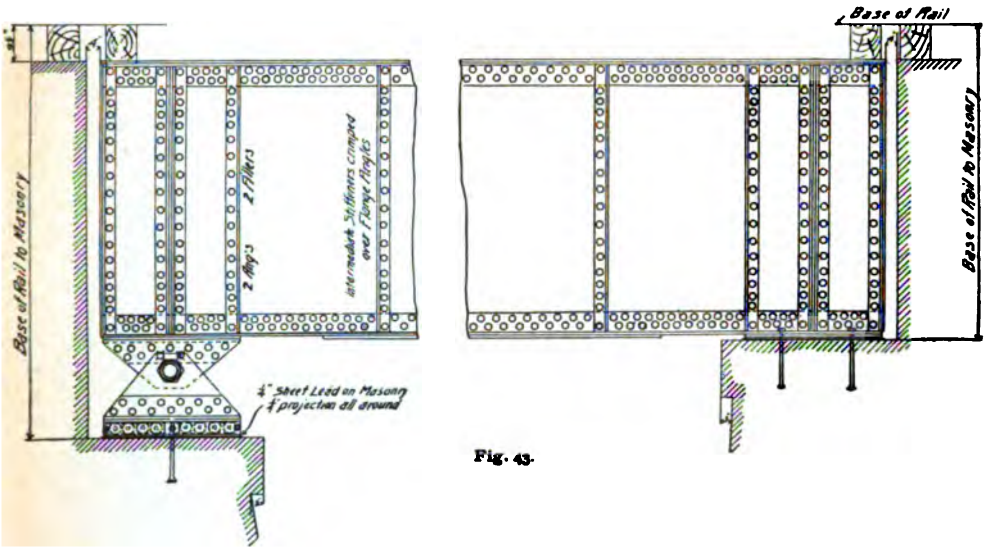
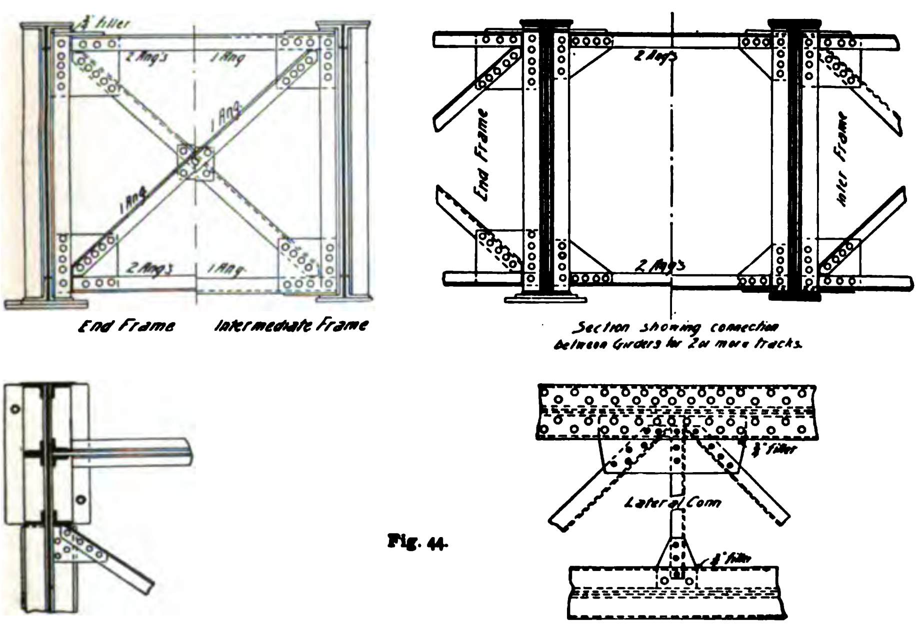


Fig. 43.



End Frame Intermediate Frame

Section showing connection between Girders for 2 or more tracks.

Fig. 44.

From Standards of New York Central and Hudson River Railroad.

The gusset plates, used as transverse bracing for the upper flanges of through plate girders will be described in Art. 45,

Fig. 45.

since in the best construction these gusset plates form an integral part of the floor system.

ART. 44. EXPANSION BEARINGS.

For short spans a base plate is riveted under the bottom flange at each end of the girder; this rests upon a bed plate that distributes the pressure to the masonry. At one end the anchor bolts hold the base plate rigidly in position, while at the other end the holes are slotted so as to permit the base plate to slide longitudinally on the bed plate under the influence of temperature changes and deflection.

When the span exceeds about 75 feet, friction rollers are introduced, and in order to insure a uniform distribution of pressure, the best practice requires at the same time the use of a pin bearing, the rollers being placed between the pedestal and the bed plate. One standard form of this bearing is shown in Fig. 46. In both shoe and pedestal, three webs composed of

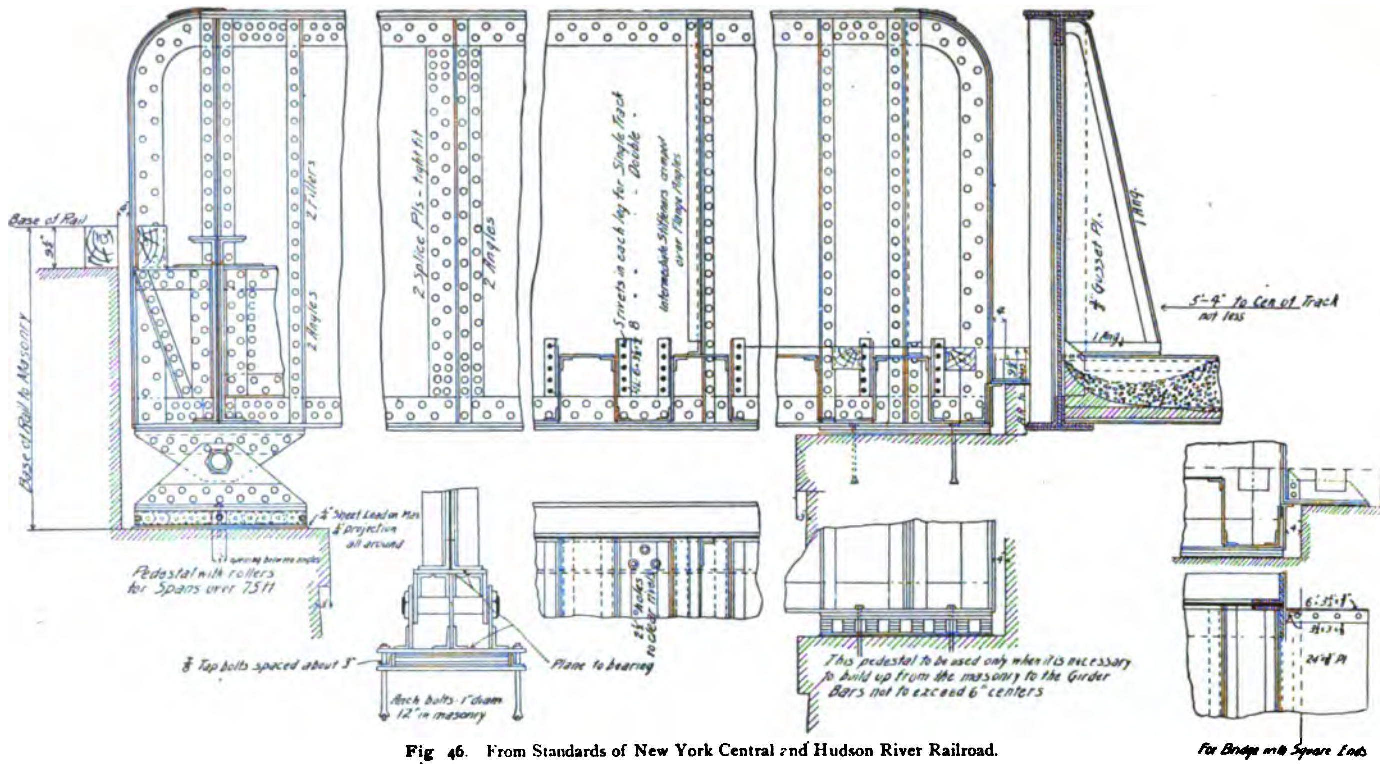


Fig 46. From Standards of New York Central and Hudson River Railroad.

vertical plates are connected to the horizontal bearing plate by means of angles. The rollers are kept at the proper clearance by two side bars which engage tap bolts in the ends of the rollers. Three tie rods hold the side bars from separating. Angle irons are placed around the nest of rollers to form a dust guard, those on the sides also acting as guides to prevent lateral motion of the girder. WADDELL specifies that the rollers are to be inclosed in dust-proof boxes filled with oil of a given quality.

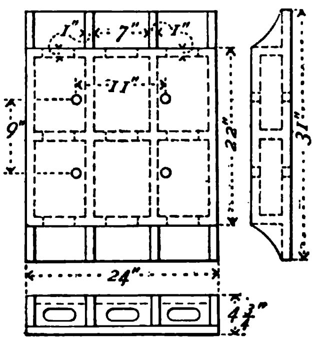
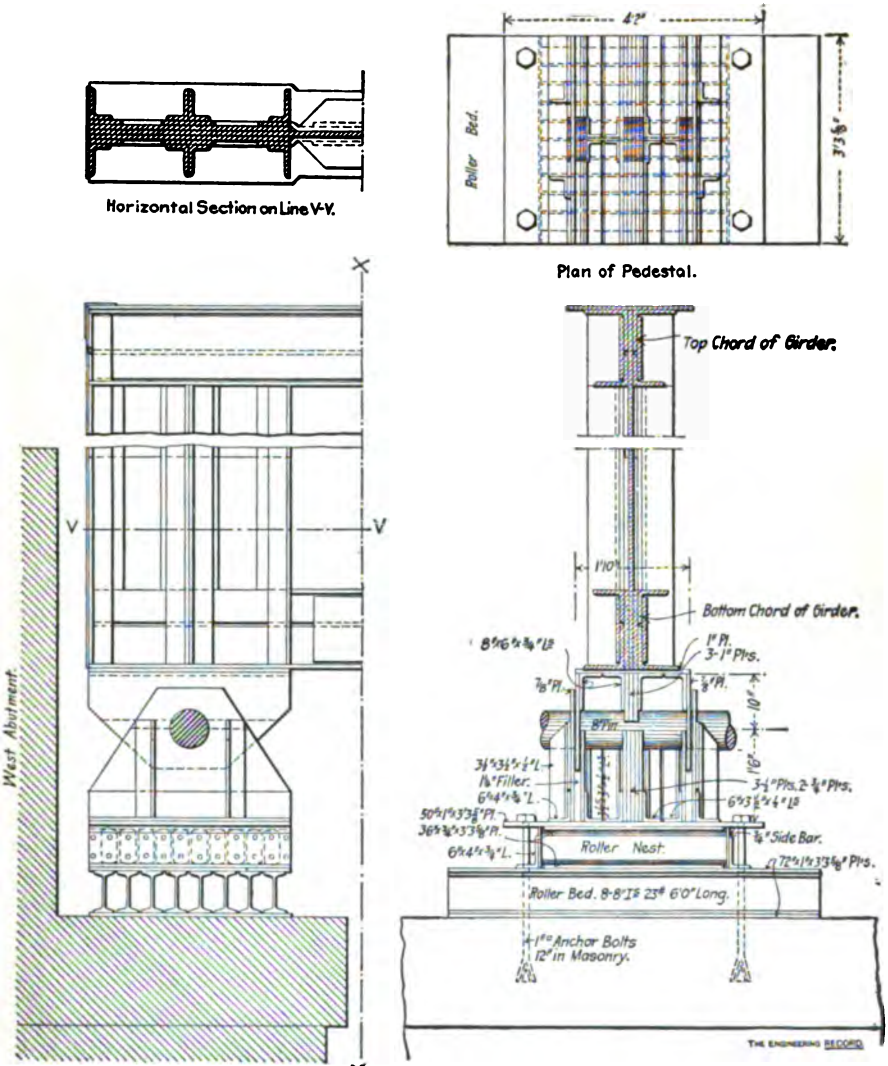


Fig. 47.

At the fixed end a cast-iron base is used whose height equals that of the rollers and bed combined so as to make the height of masonry the same at both ends of the span. One form of such a base is given in Fig. 47.

The complete detail drawings of a similar bearing designed for a span of 100 feet are given on the insets of Engineering News for July 8 and 15, 1897. The webs are stiffened by inclined diaphragms on both sides, the rollers are relatively larger, and the dust guard is differently arranged, the side bars being replaced by angles. The details of another bearing are shown in Railroad Gazette, vol. 27, page 771, Nov. 22, 1895. In this case there are only two webs, each one connected to the bearing plate with two angles. The webs are joined by a central vertical diaphragm which is arranged to take continuous bearing on the pin. The rollers move on flats riveted to the bed plate, the grooves between which gather the dust that may pass the guards and facilitate its subsequent removal. The rollers are grooved in the middle and engage a flat riveted to the base plate of the pedestal as well as the middle flat on the bed plate which is higher than



Horizontal Section on Line V-V.

Plan of Pedestal.

Elevation.

Sectional Elevation on Line X-X.

Fig. 48.

the rest, thus preventing lateral motion of the girder and relieving the side angles of the dust guard from that duty.

Fig. 48 illustrates the bearing of a 103-ton plate girder on the New York Central and Hudson River Railroad. The pedestal has five webs, a transverse diaphragm between the inner webs, the outer webs being stiffened by vertical angles on the outside. The rollers are segmental with parallel sides, and are 6″ in diameter and 3″ thick. The roller bed contains closely spaced I-beams in order to distribute the pressure on the masonry over a larger area. At the middle pier of this two-span bridge the adjacent shoes of the fixed ends of the girders engage the pins of a single pedestal whose length is not quite double that of those on the abutments. In order to secure increased web bearing on the base plate of the shoe the lower part of the web is reinforced by two plates, and the lower flange angles are crimped around them as indicated in the horizontal section.

Fig. 50 shows beds containing piles of 4″ × 1″ flats spaced 4″ apart in the clear, offset at the ends so as to secure larger bearing areas for the bed plates. The construction of the roller dust guard is shown in Fig. 49, both bearing plates above and below the rollers having $\frac{5}{16}$″ shoulders, while the side bars extend $\frac{3}{16}$″ beyond the rollers.

Fig. 51 contains the end elevation, transverse section, and side elevation of a cast-steel expansion bearing which was designed for a deck girder with a span of 112 feet. The pin has a diameter of 6 inches, and is held in place by a ring one inch thick made in two sections. The thickness of metal in the cast pedestal is 2 inches. Another bearing of the same kind is shown in Fig. 52, and was designed for a single-track through girder whose span is 125′ 2½″. The details of the steel castings indicate that the 3″ pin has continuous bearing. The pin is

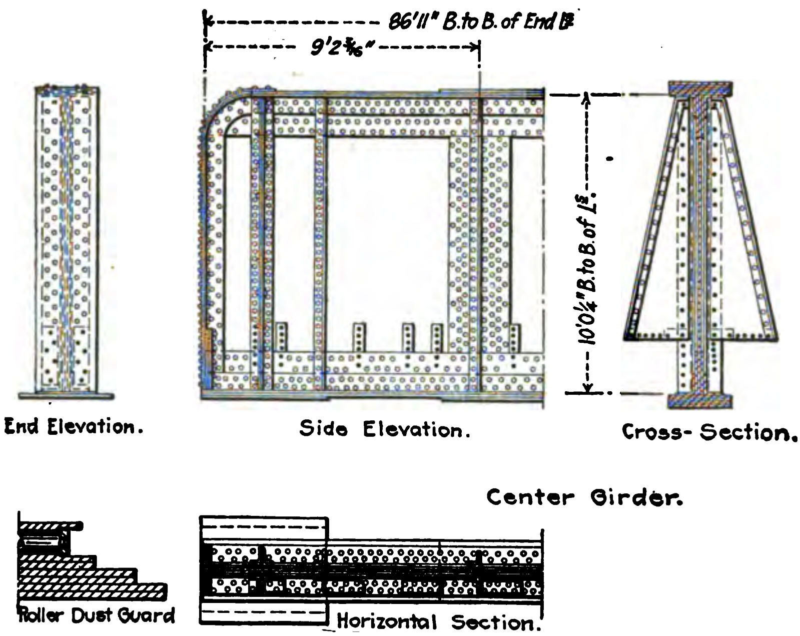

9'2"
10'0¼" B. to B. of
End Elevation.
Side Elevation.
Cross-Section.
Center Girder.
Roller Dust Guard
Horizontal Section.

Fig. 49.

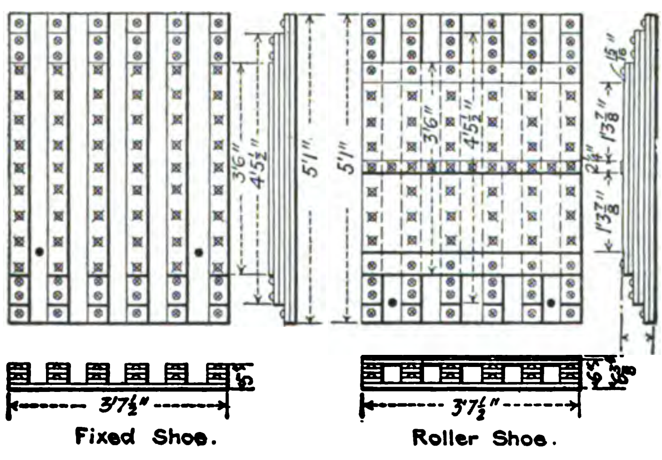

3'6"
4'5½"
5'1"
5'1"
3'7½"
3'7½"
Fixed Shoe.
Roller Shoe.
THE ENGINEERING RECORD.

Fig. 50.

omitted in the outline sketch of the assembled bearing. In Engineering Record, vol. 40, page 414, Sept. 30, 1899, may be found an illustration of a cast-steel bearing in which the pin is flattened on the lower side, while in Railroad Gazette, vol. 25, page 684, Sept. 15, 1893, another one is given in which the pedestal has a semi-cylindrical projection that enters the concave bearing of the shoe and thus replaces the pin.

A novel design was made in 1900 by the American Bridge Company, under the direction of the bridge department of the Lehigh Valley Railroad, for the support of a 116-foot deck plate-

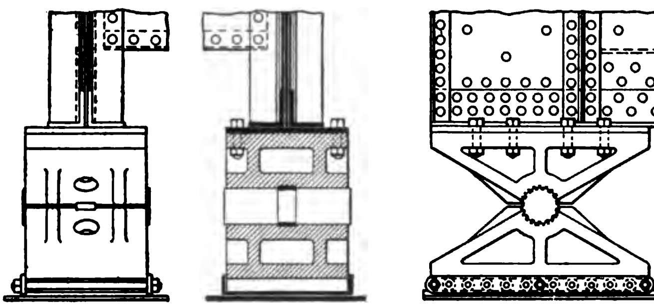


Fig. 51.

girder bridge at Mauch Chunk, Pa. As shown in Fig. 53 the end shear is transferred by pin plates directly from the web and end stiffeners to the 9-inch pin of the shoe or pedestal. By this arrangement the distance from the bottom flange of the girder to the masonry is reduced to 9 inches. The distance from the base of rail to the masonry is 10 feet $7\frac{1}{4}$ inches.

Another design embodying some unusual features is that of an expansion bearing for a plate-girder bridge of $114' 6''$ span on the Chicago, Milwaukee and St. Paul Railway at Janesville, Wis. A planed phosphor-bronze plate 8 inches square and $1\frac{1}{2}$ inches thick is inserted between the upper and lower castings, permitting the upper casting to move longitudinally and the girder to deflect without disturbing the pedestal bearings. The

castings have projecting longitudinal flanges which inclose the bronze plate and prevent lateral displacement. The plate is tap bolted to the lower casting. It will be observed that the object of these bearing plates is to replace both the pin and the friction rollers. See illustration in Engineering Record, vol. 44, page 6, July 6, 1901.

Fig. 54 shows a standard hinge joint of cast steel for spans from 50 to 65 feet, introduced on the Northern Pacific Railway. For spans over 65 feet segmental rollers 12 inches in diameter are used, the form of bearing being the same as that used for riveted trusses. See Plate II, Art. 69. These spans are considera-

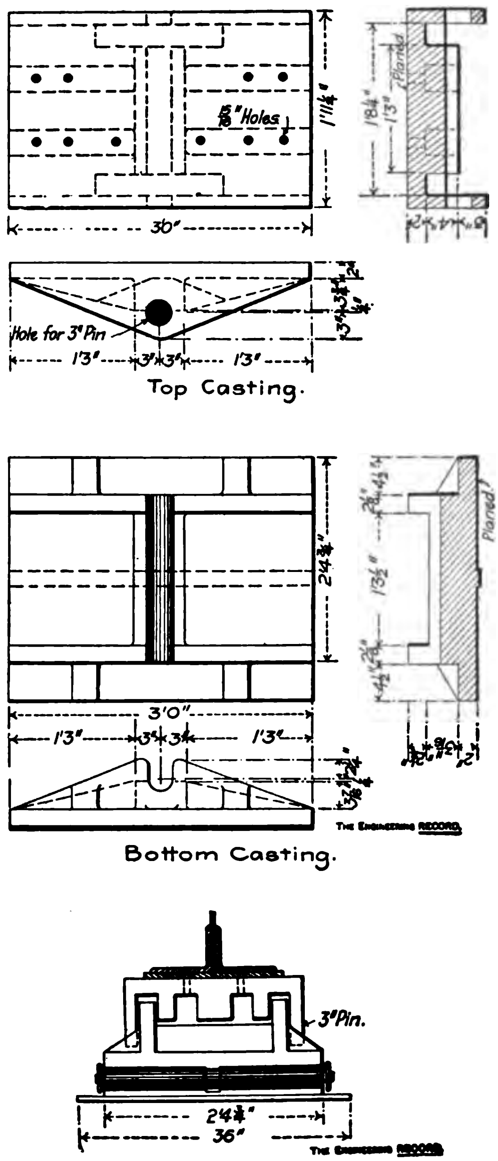
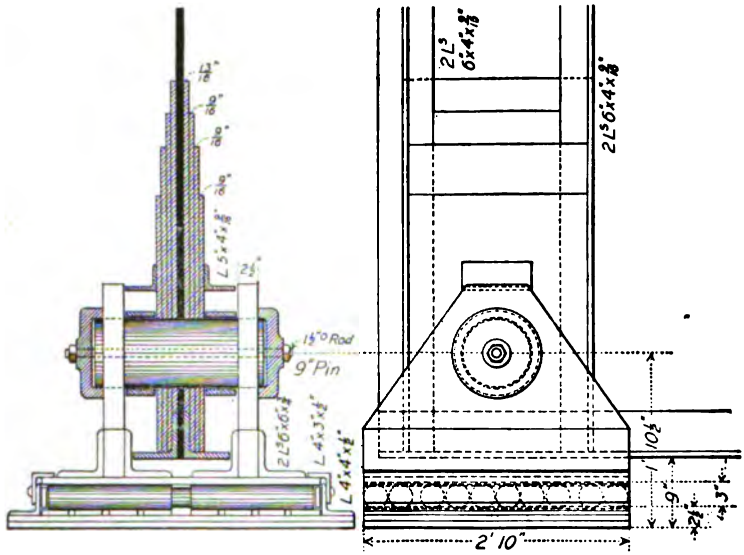
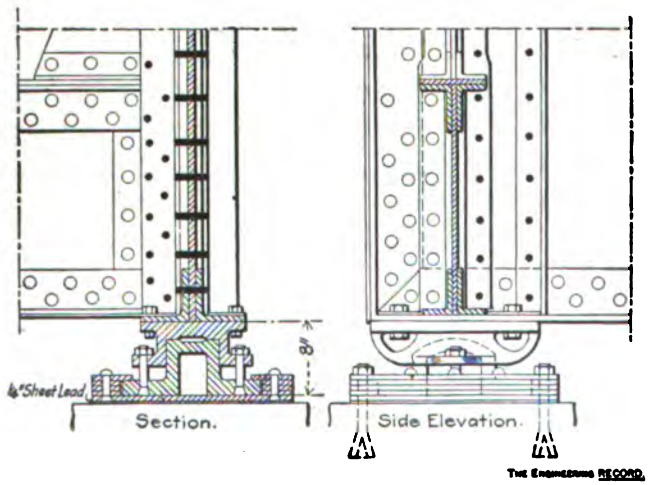


Fig. 52.



L 5"x4"x⅜"
9" Pin
2' 10"
10½"

Fig. 53.



Section.
Side Elevation.
8"
THE ENGINEERING RECORD.

Fig. 54.

bly less than the usual limit assigned in practice to the use of expansion bearings. The details of a rocker bearing, formerly a standard on the same railroad, may be seen in Engineering Record, vol. 37, page 5, Dec. 4, 1897. The rocker consists practically of a double pin 4″ in diameter and 8″ high.

ART. 45. FLOOR SYSTEM.

The floor system of through railroad plate-girder bridges in most extensive use consists of floor beams and stringers, the

Fig. 55.

latter supporting the cross-ties on their upper flanges. The floor beams are generally spaced from 12 to 18 feet apart. The spacing is slightly less in single-track than in double-track bridges. The general arrangement of the floor system and its connection with the girders is clearly shown in Fig. 55, the view being taken before one of the tracks was put in place. The bridge is on the Baltimore and Ohio Railroad.

A floor beam is in reality a plate girder of short span, as indicated in Fig. 56. This illustration is a part of the standard details adopted by the New York Central and Hudson River Railroad. The web is spliced near each end in order to make an efficient connection with the main girders, both to transfer

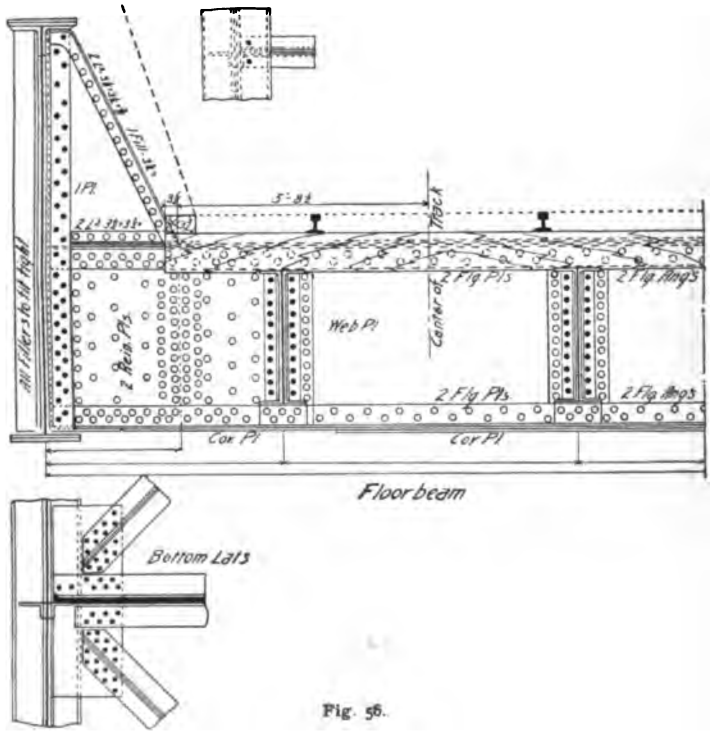


Fig. 56.

the shears and to stiffen the upper flange of the girders. As shown in the figure the triangular gusset plate is an upward extension of the end web plate, while the web splice plates are extended from the girder to the inside of the nearer stringer, to reinforce the web plate. This reinforcement is not needed in a single-track bridge. The outer edge of the gusset plate is stiffened by a pair of angles whose upper ends are bent over and

riveted to both the stiffeners and flange angles of the girder. This is more effective than the common arrangement in which these angles are cut off where they meet the edge of the stiffeners. Sometimes the gusset plate is made too narrow and not even extended up to the flange angles as it should be, while in other cases this plate is separate from the web plate of the floor beam, and only connected to it indirectly by short horizontal angles riveted to the flanges. Such a connection develops tension in some of the rivets, which should be avoided when possible.

When the lower flange of the girder contains four angles, and it is desirable to keep the bottom of the floor beam as low as

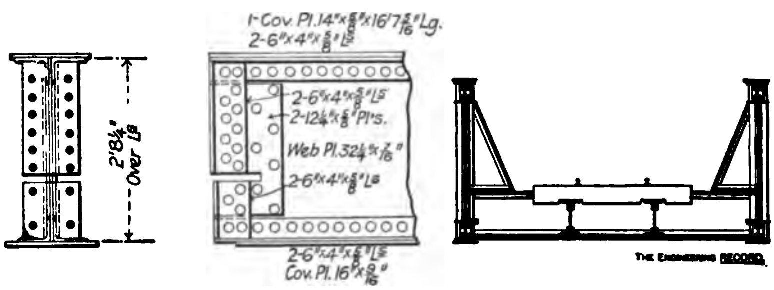
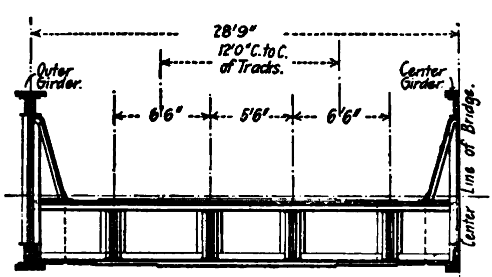


Fig. 57.

possible, the end of the floor beam must either be slotted, as in Fig. 57, or the connection to the girder may be omitted below the top angles of its lower flange, as in Fig. 58, since some of the shear may be transferred to the stiffeners above the floor beam, on account of the continuity of the end web plate.

The stringers may consist either of I-beams or be built up like a plate girder, in which case it is preferable to use no cover plates, and to allow the web plates to project above the flange angles. The left end of Fig. 46, Art. 44, shows a bracket, in line with the stringer and beyond the end floor beam, which carries the end track tie on the bridge.

In another arrangement of the floor system of through plate-girder bridges the ties rest on horizontal shelf angles riveted to the web near the lower flanges, and the gusset plate stays are attached to the transverse struts of the lateral system, which are made as deep as the track will allow, in order to give the needed stiffness. The method is objectionable on account of the warping of the timber, in spite of all precautionary appliances to prevent it, while at the same time the cross-ties are liable to be pushed over in case of derailment, on account of their greater depth required by the increased span of the ties. It is worse still to rest the cross-ties on the bottom flanges, since the constant springing of the floor tends to weaken the flanges and to loosen the lateral bracing.



Half Transverse Section.

THE ENGINEERING RECORD.

Fig. 58.

In deck bridges the cross-ties rest directly upon the top flanges of the girders, two girders supporting each track. (See Fig. 43 in Art. 43.) This is the plan almost universally adopted. On the Boston and Maine Railroad, however, the standard deck-girder bridge contains floor beams and stringers. The floor beams are riveted to the webs of the girders at such a height that the stringers, which rest on top of them, are about even with the tops of the girders. The cross-ties extend over both stringers and girders, the latter, spaced 9 feet apart, acting thus also as safety stringers. There is but one lateral system, it being in the plane of the bottom flanges of the floor beams. The upper flanges of the girders are stayed by transverse web connections to the stringers directly over the floor beams.

A similar arrangement is adopted in the long span deck-girder bridges of the approaches to the Monongahela river bridge at Rankin, Pa. The track is on a curve, requiring the girders to be spaced 17′ 3″ or more apart. The stringers are 6′ 6″ apart and follow the track as nearly as possible, the track running close to the inner girder at its ends and the outer girder at its center.

In highway bridges floor beams are generally used, and to these are attached steel I-beams or wooden joists spaced relatively close together to give adequate support to the wooden floor planks or to the buckle plates which carry some form of paved floor. The sidewalks outside of the girders are carried on brackets or cantilever extensions of the floor beams. In the Journal of the Association of Engineering Societies, vol. 21, page 62, Aug., 1898, an illustration is shown of some grade-crossing work, in which the floor beams are dropped down below the lower flanges of the girders about half their depth in order to save head room over the railroad tracks below. The floor beams are placed midway between the clearances required by the railroad.

A large number of standard railroad bridge floors for deck and through bridges are described and illustrated by 14 plates, showing partly dimensioned details, in the report of a Committee of the Association of Railway Superintendents of Bridges and Buildings, published in BERG's American Railway Bridges and Buildings, pages 645–669, reprinted from the Proceedings of the Association.

ART. 46. SOLID FLOORS.

Solid floors in railroad bridges include many different types of continuous metal floors which support the rails on ordinary cross-ties in ballast. In some cases the ballast is omitted, and the cross-ties rest on the metal floor, while in other cases the

cross-ties are also omitted, and the rails are bolted directly to the metal floor.

In the earliest type used in this country old track rails were laid close together on top of the girders of short span bridges, and on which the ballast was spread, thus securing a continuous track free from the objections inhering in the transition to and from the standard wooden bridge floor supported on stringers. Floors of this kind were built as early as 1874.

The next form, which was introduced in 1887, consists of metal troughs built by riveting trough plates that are alternately inverted, as indicated in Fig. 59. The cross-ties are sometimes laid directly in the troughs, but more frequently bedded in ballast. In the following year the New York Central and Hudson River Railroad began building solid floors with continuous

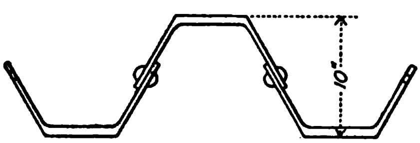


Fig. 59.

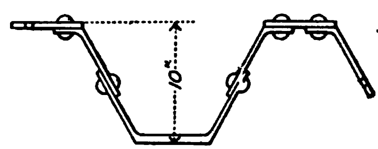


Fig. 60.

ballast, adopting the above section for deck bridges and the rectangular trough section, consisting of plates and angles (Fig. 64), for through bridges. The latter section is better adapted to being hung between the girders by connecting plates and angles than any form having inclined sides, and its depth may be readily increased to give the required strength for any given load and span. (See Fig. 46 in Art. 44.) This railroad was the first to adopt solid floors as a standard, and has continued to the present time to use the rectangular troughs with ballasted track where the depth of floor is limited.

Fig. 60 gives a trough section composed of a flat, an obtuse angle, and a trough plate, which was designed to be easily manufactured and power riveted, adjustable for depth and spacing,

absolutely water tight, and to avoid rivets in the tension side with the consequent reduction in strength. The sections in Figs. 61 and 62 have a fixed depth, but are readily adjustable horizontally by varying the widths of the flats, the latter re-

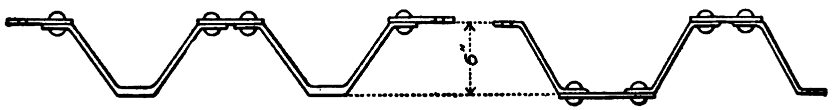


Fig. 61. Fig. 62.

quiring twice as much riveting as the former, half of it being in the bottom of the troughs.

The triangular form in Fig. 63 has been used to a limited extent, the lower edge not being well adapted for support, while the inclined sides render web connections with the girders less convenient. Fig. 65 shows a modification of the rectangular trough in which Z-bars take the place of the angles and vertical plates of Fig. 64.

The requirements of track elevation in large cities for very shallow floors in the plate-girder bridges at street crossings have led to several designs of floors composed of I-beams and continuous cover plates. In some cases the transverse I-beams

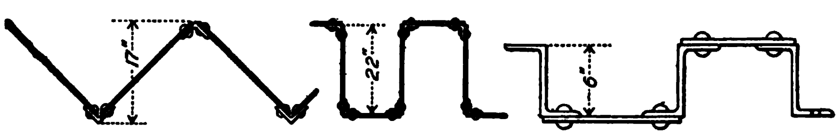


Fig. 63. Fig. 64. Fig. 65.

are spaced so close together as not to require any stringers, while in others they are spaced somewhat farther apart and short stringers are used to support the rails.

In highway bridges those floors are called solid floors in which there is either a continuous metal floor, or in which the metal and some other material, like concrete, for example, form

the permanent foundation to receive some kind of street and sidewalk surface paving.

The form most generally used consists of buckled plates supported on stringers and floor beams properly spaced, upon which concrete is placed to receive the rest of the pavement. Buckled plates were introduced in this country in 1875. Plain and corrugated or arched plates are also used in a similar manner, while some of the types of solid floors employed in railroad bridges have been adopted for highway bridges.

The great lateral strength of all solid floors renders a lateral system unnecessary for bridges containing them.

Solid floors composed of reinforced-concrete slabs supported by closely spaced I-beams were introduced by the Wabash Railroad in 1903. Concrete had been used previously to some extent as filling for the protection of metal in steel trough floors, and when I-beams came to be more generally adopted instead of troughs, the continuous steel plate which was at first laid over the beams to support the ballast, was replaced by a filling of plain concrete between the beams and extended over their upper flanges to protect them. The satisfactory service given by the reinforced-concrete slab floors soon led to their adoption by other railroads and for highway bridges, with various modifications in details and with increasing spans.

References to engineering periodicals are given in Art. 47 which relate to illustrated descriptions of various kinds of solid floors constructed since 1900, and arranged chronologically to indicate more clearly the order of development. In the first two groups the articles are confined practically to the floor system, whether they are used in beam, plate-girder, or truss bridges. In Art. 48 are given selected references to articles containing descriptions and illustrations of solid floors, and also of other details of the plate-girder bridges in which they occur.

The references to engineering periodicals from 1889 to 1901 inclusive, extending back to the introduction of solid floors for bridges in America, which were given in the previous edition of this book, have been omitted in the present edition, except those for 1901.

ART. 47. SOLID BRIDGE FLOORS — REFERENCES.

Solid-floor Single-track Plate-girder Bridge. Eng. Rec., v. 46, p. 152, Aug. 16, 1902.

Protection of Steel in Ballasted Floor Bridges. Eng. News, v. 50, p. 437, Nov. 12, 1903.

Best Methods of Protecting Solid Steel Floors of Bridges. Report of Committee and Discussions. Proc. Assoc. Ry. Supt. Bridges and Bldgs., 1903, v. 13, p. 149.

Fireproof Floor of the Williamsburg Bridge, New York. Eng. Rec., v. 49, p. 466, Apr. 9, 1904.

Waterproofing Bridge Floors on the Chicago and Western Indiana R. R. Eng. News, v. 51, p. 440, May 5, 1904.

Ballasted Railroad Bridge with a Plank Floor. Eng. Rec., v. 49, p. 643, May 21, 1904.

Ballasted Floor Through-span Bridge for the Santa Fé. R. R. Gaz., v. 36, p. 394, May 27, 1904.

Double-track Pin-connected Railroad Bridge with Stringer and Solid Plate Flooring. Eng. Rec., v. 51, p. 324, Mar. 18, 1905.

Oak Lane Bridge, Reading Railway, Philadelphia. Eng. Rec., v. 51, p. 581, May 20, 1905.

Ballasted Bridge Floors. By A. F. ROBINSON. Jour. W. Soc. Engrs., v. 10, p. 227, June, 1905.

Protecting and Waterproofing Solid-floor Bridges. By W. C. CUSHING. R. R. Gaz., v. 39, p. 104, Aug. 4, 1905.

Thirty-three-track Bridge at Chicago. Eng. Rec., v. 53, p. 731, June 16, 1906.

Bridges on the Fortieth Street Line of the Chicago Junction Railway. Eng. Rec., v. 54, p. 209, Aug. 25, 1906.

Erection and Waterproofing of Plate-girder Bridges at Plainfield, N. J. Eng. Rec., v. 57, p. 134, Feb. 1, 1908.

Waterproofing Ballasted Bridge Floors at Schenectady, N. Y. Eng. Rec., v. 57, p. 371, Mar. 28, 1908.

Open vs. Ballasted Deck Structures. By A. F. ROBINSON. Proc. Am. Ry. Eng. & M. W. Assoc., 1908, v. 9, p. 253.

Waterproofing of Concrete-covered Steel Floors of Bridges. Report of Committee and Discussions. Proc. Assoc. Ry. Supt. Bridges and Bldgs., 1908, v. 18, p. 46. Abstract in Eng. Rec., v. 58, p. 488, Oct. 31, 1908.

Ballasted Floors on Erie Railroad Bridges. Eng. Rec., v. 59, p. 250, Feb. 27, 1909.

Solid-floor Short-span Railroad Bridges. Eng. Rec., v. 59, p. 548, Apr. 24, 1909.

Chicago Track Elevation. By M. K. TRUMBULL. Ry. Age Gaz., v. 46, p. 1175, June 4, 1909.

Twelfth Street Bridge, Philadelphia, Pa. Eng. Rec., v. 59, p. 740, June 12, 1909.

Construction of the Four-track Truss Bridge with Solid Floor, Chicago and Oak Park Elevated Railway. Eng. News, v. 62, p. 653, Dec. 6, 1909.

Heavy Floor for a Double-track Bridge. Eng. Rec., v. 61, p. 76, Jan. 15, 1910.

Progress Report upon Waterproofing Masonry. Proc. Am. Ry. Eng. Assoc., 1910, v. 11, p. 970. Abstract in Eng. Rec.,

v. 61, p. 361, Mar. 26, 1910; Ry. Age Gaz., v. 49, p. 212, Aug. 5, 1910. See also reports in later volumes of the Proceedings.

REINFORCED-CONCRETE FLOORS.

Steel-concrete Abutments and Solid Floors for Railroad Bridges. By A. O. CUNNINGHAM. R. R. Gaz., v. 36, p. 20, Jan. 8, 1904.

Solid Floor for Deck Girders. R. R. Gaz., v. 36, p. 114, Feb. 12, 1904.

Reinforced-concrete Floor for Deck Girders. R. R. Gaz., v. 38, p. 96, Feb. 3, 1905.

Concrete Floors for Railway Bridges. Eng. News, v. 53, p. 161, Feb. 16, 1905.

Shallow Solid-floor Girder. R. R. Gaz., v. 38, p. 365, Apr. 21, 1905.

Test of Concrete Bridge Floors. Eng. Rec., v. 52, p. 104, July 22, 1905.

Reinforced-concrete Structures for Railroads. By A. O. CUNNINGHAM. Eng. Rec., v. 52, p. 491, Oct. 28, 1905.

Proposed Concrete Floors for Railway Bridges and Tracks. By J. W. SCHAUB. Eng. News, v. 54, p. 460, Nov. 2, 1905; R. R. Gaz., v. 39, p. 426, Nov. 3, 1905.

Reinforced-concrete Floors for Through-truss Spans. R. R. Gaz., v. 42, p. 339, Mar. 15, 1907.

Solid Floor for Plate-girder Spans. Eng. Rec., v. 56, p. 185, Aug. 17, 1907.

Double-track Work through Eagle River Canyon, Denver and Rio Grande Railroad. Eng. Rec., v. 56, p. 533, Nov. 16, 1907.

Waterproofing a Solid-floor Bridge. Eng. Rec., v. 59, p. 77, Jan. 16, 1909.

Protected Steel and Concrete Highway Bridge. Eng. Rec., v. 59, p. 204, Feb. 20, 1909.

Steel Highway Bridge with Concrete Stringers and Floor Slabs. Eng. Rec., v. 59, p. 334, Mar. 20, 1909.

Big Four Track Elevation at Indianapolis. Ry. Age Gaz., v. 46, p. 890, Apr. 23, 1909.

Special Track Elevation Bridges in Chicago. Eng. Rec., v. 60, p. 549, Nov. 13, 1909.

SOLID FLOORS IN BEAM BRIDGES.

Improvements on the Cleveland Division of the Baltimore and Ohio Railroad. R. R. Gaz., v. 38, p. 220, Mar. 17, 1905.

Kansas City Topeka Double-track Work of the Union Pacific. R. R. Gaz., v. 39, p. 540, Dec. 8, 1905.

Short-span Bridges on the Baltimore and Ohio Railroad. Eng. Rec., v. 53, p. 744, June 16, 1906.

Chicago Terminal Transfer Track Elevation. Ry. Age Gaz., v. 45, p. 1052, Oct. 2, 1908.

Standard I-beam Railroad Bridges over City Streets. Eng. Rec., v. 62, p. 316, Sept. 17, 1910.

Kensington Avenue Bridge, Buffalo. Eng. Rec., v. 62, p. 465, Oct. 22, 1910.

Protected I-beam Bridge over Railroad Tracks. Eng. Rec., v. 64, p. 67, July 15, 1911.

ART. 48. SOLID FLOORS IN PLATE-GIRDER BRIDGES — REFERENCES.

The following references relate to descriptions and illustrations of various types of solid floors in plate-girder bridges for both

railroads and highways. For convenience, those on reinforced-concrete floors are grouped together.

Three-track Beam-floor Girder Bridge. Eng. Rec., v. 49, p. 770, June 18, 1904.

Southwestern Avenue Boulevard Nine-track Viaduct in Chicago. Eng. Rec., v. 51, p. 484, Apr. 29, 1905.

Chew Street Double-track Plate-girder Bridge at Walnut Lane. Eng. Rec., v. 51, p. 498, Apr. 29, 1905.

Wabash River Bridge at Terre Haute, Ind. By M. A. HOWE. Eng. News, v. 55, p. 273, Mar. 8, 1906.

Special Details of a Long Plate-girder Span. Eng. Rec., v. 53, p. 718, June 9, 1906.

Short-span Bridges on the Baltimore and Ohio Railroad. Eng. Rec., v. 53, p. 744, June 16, 1906.

Grade-crossing Removal of the Philadelphia and Reading Railroad in the City of Philadelphia. Eng. Rec., v. 58, p. 48, July 11, 1908; Eng. News, v. 60, p. 96, July 23, 1908; Ry. Age Gaz., v. 45, p. 612, July 31, 1908.

Short-span Bridges over City Crossings. Eng. Rec., v. 60, p. 50, July 10, 1909; p. 74, July 17, 1909.

Six-track Short-span Solid-floor Bridge. Eng. Rec., v. 60, p. 109, July 24, 1909.

Harlem River Branch Improvements, New York, New Haven and Hartford. Ry. Age Gaz., v. 48, p. 186, Jan. 28, 1910; p. 257, Feb. 4, 1910.

Protected Plate-girder Railroad Bridge. Eng. Rec., v. 63, p. 671, June 17, 1911.

Double-track Skew Bridge with Solid Floor. Eng. Rec., v. 64, p. 226, Aug. 19, 1911.

REINFORCED-CONCRETE FLOORS.

Forest Park Bridge of the Wabash at St. Louis. R. R. Gaz., v. 37, p. 488, Oct. 28, 1904; Eng. Rec., v. 50, p. 549, Nov. 5, 1904; Eng. News, v. 52, p. 431, Nov. 17, 1904.

Lawrence Street Bridge, Denver. Eng. Rec., v. 52, p. 638, Dec. 2, 1905.

Colfax Avenue Bridge, South Bend, Ind. Eng. Rec., v. 53, p. 793, June 30, 1906.

Bay Ridge Improvement Bridges. Eng. Rec., v. 54, p. 181, Aug. 18, 1906.

Reinforced-concrete Floor for Plate-girder Deck Bridges, C. C. C. & St. L. Ry. Eng. News, v. 57, p. 539, May 16, 1907.

Six-track Solid-floor Plate-girder Bridge. Eng. Rec., v. 55, p. 728, June 22, 1907.

Rex-red Cliff Double-track Construction on the Denver & Rio Grande R. R. Eng. News, v. 58, p. 543, Nov. 21, 1907.

Sixth Street Viaduct, Kansas City. By E. E. HOWARD. Trans. Am. Soc. C. E., v. 65, p. 42, Dec., 1909.

109-foot Concrete-floor Plate-girder Bridge. Eng. Rec., v. 61, p. 662, May 21, 1910.

Kensington Avenue Bridge, Buffalo. Eng. Rec., v. 62, p. 465, Oct. 22, 1910.

Walpole Bridge. Eng. Rec., v. 62, p. 545, Nov. 12, 1910.

Calvin Street Bridge, Buffalo. Eng. Rec., p. 62, p. 732, Dec. 24, 1910.

ART. 49. PLATE-GIRDER BRIDGES — REFERENCES.

The following references relate to descriptions and illustrations of the characteristic features of plate-girder bridges con-

structed since 1900 and which have the ordinary type of open floor. A careful study of these articles will familiarize the student with many important features of recent practice in plate-girder design and construction.

DECK RAILROAD BRIDGES.

Northern Pacific Standard Bridge Plans. By RALPH MODJESKI. Jour. W. Soc. Engrs., v. 7, p. 51, Feb., 1901.

Janesville Bridge. Eng. Rec., v. 44, p. 6, July 6, 1901.

Standard Plans for Bridges on the Atchison, Topeka & Santa Fé Ry. Eng. News, v. 49, p. 482, May 28, 1903; Eng. Rec., v. 48, p. 598, Nov. 14, 1903.

Large Plate-girder Bridge on the Erie. R. R. Gaz., v. 36, p. 345, May 6, 1904; Eng. News, v. 51, p. 166, Feb. 18, 1904; Eng. Rec., v. 52, p. 324, Sept. 16, 1905.

100-foot Plate-girder Span with Unusual Bearings. Eng. Rec., v. 51, p. 46, Jan. 14, 1905.

Standard Bridges on the Harriman Lines. R. R. Gaz., v. 38, pp. 248, 278, 310, 328, 347, 370, 389, Mar. 17, 24, 31, Apr. 7, 14, 21, 28, 1905.

Pine Creek Long-span Plate-girder Bridge. Eng. Rec., v. 51, p. 530, May 6, 1905.

Some Deck Plate-girder Bridges on the New York, New Haven & Hartford R. R. Eng. Rec., v. 52, p. 386, Sept. 30, 1905.

Standard Plate Girders on the Chicago, Milwaukee & St. Paul Ry. Eng. Rec., v. 53, p. 74, Jan. 20, 1906.

122-foot Four-track Plate-girder Span. Eng. Rec., v. 53, p. 605, May 12, 1906.

Warehouse Point Bridge of the New Haven R. R. Eng. Rec., v. 55, p. 520, Apr. 27, 1907.

Long Plate-girder Bridge, Towanda, Pa. Eng. Rec., v. 56, p. 522, Nov. 9, 1907; Eng. News, v. 59, p. 113, Jan. 30, 1908.

Shenango River Bridge, Pittsburg & Lake Erie Railroad. Eng. Rec., v. 59, p. 155, Feb. 6, 1909.

THROUGH RAILROAD BRIDGES.

Northern Pacific Standard Bridge Plans. By RALPH MODJESKI. Jour. W. Soc. Engrs., v. 7, p. 51, Feb., 1901.

A 103-ton Plate Girder. Eng. Rec., v. 43, p. 102, Feb. 2, 1901.

Standard Plans for Bridges on the Atchison, Topeka & Santa Fé Ry. Eng. News, v. 49, p. 482, May 28, 1903; Eng. Rec., v. 48, p. 598, Nov. 14, 1903.

Bridge on Yellow Creek; Lake Erie, Alliance & Wheeling Ry. Eng. News, v. 51, p. 168, Feb. 18, 1904.

Improvement of a Plate-girder Bridge at Hartford, Conn. By H. R. BUCK. Eng. News, v. 51, p. 276, Mar. 24, 1904.

Plate-girder Approaches of the Clairton Bridge. Eng. Rec., v. 49, p. 383, Mar. 26, 1904.

Long-span Double-track Plate-girder Bridge. Eng. Rec., v. 49, p. 491, Apr. 16, 1904.

Bethlehem Junction Bridge. Eng. Rec., v. 51, p. 68, Jan. 21, 1905.

Some Through Plate-girder Bridges, New York, New Haven & Hartford R. R. Eng. Rec., v. 52, p. 407, Oct. 7, 1905.

Standard Plate Girders on the Chicago, Milwaukee & St. Paul Ry. Eng. Rec., v. 53, p. 74, Jan. 20, 1906.

Plate-girder Spans on the Chicago, Burlington & Quincy R. R. Eng. Rec., v. 53, p. 191, Feb. 17, 1906.

Shallow Double-track Bridge Floor. Eng. Rec., v. 63, p. 307, Mar. 18, 1911.

RAILROAD AND HIGHWAY BRIDGES.

Types and Details of Bridge Construction. Part II. Plate Girders. Examples of Constructed Railroad and Highway Spans. By FRANK W. SKINNER. New York, 1906. 412 pages with numerous illustrations of details.

NOTE. — Pages 144 and 145 are left blank.

CHAPTER VII.

DESIGN OF A PLATE-GIRDER BRIDGE.

ART. 50. SPECIFICATIONS.

Let it be required to design a deck plate-girder bridge for a single-track railroad, the span being 80 feet between centers of supports. The bridge is to be located on a straight track, and its material, with the exception of the rivets and the track, is to be medium steel.

In order to avoid the inconvenience to the student of continually referring to other pages while following the computations required for the design, the specifications will not be printed separately, but will be given in the text as needed. That the

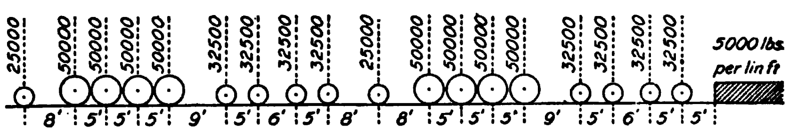


Fig. 66.

student may gain some familiarity with the range of variation in modern specifications relating to some of the details, according to a number of the leading standard specifications for steel railroad bridges, references will sometimes be made to such variations before adopting the respective requirements.

The live load is to consist of two consolidation locomotives and a train, as shown in Fig. 66, or an alternative load of 120 000 pounds, equally distributed on two pairs of driving wheels, spaced 6 feet center to center. The above loading is known as COOPER'S Standard, Class E 50, and equals the 1900 standard of the Lehigh

Valley Railroad for its main line, with the exception of the alternative load, which is slightly different. It is also the 1901 standard of the Baltimore and Ohio Railroad.

The allowance for impact due to the live load is that recommended by WADDELL, the coefficient of impact being

$$I = 400/(L + 500),$$

in which L is the length in feet of that portion of the span which is covered by the live load when the maximum stress under consideration is produced, and I is the percentage by which the maximum live load stress is to be increased.

ART. 51. DEPTH AND SPACING.

A comparison of a number of recent designs of plate girders for some of the principal railroads shows a variation in the depth from one-tenth to one-twelfth of the span. In one case, where the span is over 125 feet, the depth is only one-fourteenth of the span; and in a few others the limitations imposed by track elevation led to the adoption of the same relative depth. There are also a few cases of very short spans in which the depth is one-eighth or one-ninth of the span. The average ratio of the depth to the span is found to be the reciprocal of 10.5.

Some specifications state that the depth of a plate girder shall preferably be not less than one-tenth of the span, and others substitute the fraction of one-twelfth, while many make no reference to the subject.

An interesting formula, deduced by HENRY SZLAPKA, for finding the economic depth of plate girders, was published in Engineering Record, vol. 35, page 363, March 27, 1897. The economic depth was found to vary from one-seventh to one-ninth of the span under the conditions and assumptions mentioned in the article.

The spacing of deck girders, center to center, ranges from 5 to 9 feet. Comparatively few are spaced less than 6½ feet apart, and the larger girders are spaced in proportion to their depth. WADDELL's rule makes it the nearest even foot to one-tenth of the span. Although in some deep girders the spacing slightly exceeds the depth, in general it is somewhat less, the difference rarely exceeding 6 inches. For the design in this chapter let the depth of the web plate be taken as 7 feet and the spacing of the girders as 8 feet. A discussion of the economic depth may be found in Art. 66.

Through girders must be spaced so as to provide the necessary clearance specified, although in track elevation, where the bridges must carry many tracks at the standard distance of 13 feet apart between center lines, they have been spaced at the same distance. The usual spacing ranges from about 14½ feet for short spans to 17 feet for long spans for a single-track bridge.

ART. 52. THE WOODEN FLOOR.

The floor usually consists of cross-ties of rectangular section, notched at least ½ inch over the flange of each supporting girder, certain ties being secured to each flange by a ¾-inch hook bolt. Practice varies by making this attachment apply respectively to every tie, to alternate ties, or to every third, fourth, or fifth tie. The space between cross-ties is not generally to exceed 6 inches nor to be less than 5 inches.

Outer guard timbers 6 by 8 inches are laid flat and parallel to the track rails and notched one inch over every cross-tie, to hold them in their relative positions longitudinally. Their attachment to the cross-ties by ¾-inch bolts varies in practice in the same manner as that noted above for securing the ties to the girders. The guard timbers are spliced over a tie by a half-lap joint 6 inches long, and a bolt must be passed through the

splice to secure the ends of both timbers to the tie. The inner face of the guard timber is placed anywhere between 11 inches from the gage side of the rail head to 5 feet 4 inches from the center of the track. It should be placed near the end of the standard 12-foot tie.

Sometimes inner guard timbers are placed with a clearance of 6 to 10 inches between them and the rail heads, either with or without angle irons to protect the outer edges, but more frequently old track rails are employed as the inner or true guard rails.

The alternative loading given in Art. 50 causes the greatest stress in the cross-ties. The load on one wheel is 30 000 pounds, and it is customary to regard this load as distributed over three cross-ties. The live load for one tie is therefore 10 000 pounds and the impact 8000 pounds. Assuming the weight of the track at 450 pounds per linear foot, and that one tie will carry a length of track of $1\frac{1}{4}$ feet, it will be sufficiently precise to regard this entire load as concentrated at the track rails. The cross-tie then acts as a beam whose supports are 96 inches apart and carrying two equal and symmetrically placed concentrated loads $59\frac{1}{2}$ inches apart, each of which is 18 280 pounds.

For a long-leaf yellow pine cross-tie a unit stress of 2000 pounds per square inch in the outer fiber may be taken when the effect of impact is considered. If b be the breadth, and d the depth of the cross-tie, and the bending moment is equated to the resisting moment, both being expressed in pound-inches, there follows

$$333\,600 = 2000\,bd^2/6,$$

whence $bd^2 = 1000$. To determine the breadth, let the safe bearing on the side of the fiber be taken at 400 pounds per square inch. The bearing area required is then $18\,280/400 = 45.7$ square inches, and if the width of the base of the rail

be 6 inches, the breadth b must be 8 inches. The net depth is therefore 11.2 inches, making the gross depth 11.7 or 12 inches. Since the notch in the cross-tie is so near the section under the track rail, only the net depth can be used in computing the strength under flexure. On account of the variation in the total thickness of cover plates, cross-ties 13 inches deep may be required near the ends of the girder.

Let the cross-ties be spaced 6 inches in the clear, and every alternate one bolted to the girder flange and the wooden guard rail respectively. At $3\frac{3}{4}$ pounds per foot, board measure, the weight of one tie 12 feet long is 360 pounds. This makes the weight of the cross-ties 310 pounds per linear foot. The weight of rails, splices, guard rails, bolts, spikes, etc., will be assumed at 160 pounds per linear foot.

ART. 53. WEB SECTION.

SPECIFICATION. — The rivets used shall be seven-eighths of an inch in diameter. The shearing stress in web plates shall not exceed 12 000 pounds per square inch; but no web plate shall be less than $\frac{3}{8}$ inch in thickness.

According to the method explained in Part II, Art. 43, the maximum live-load shear is found to be 155 100 pounds, and from the formula for the coefficient of impact given in this chapter, Art. 50, the corresponding impact allowance is 107 000 pounds.

The net weight of one girder and one-half of the lateral bracing and cross-frames will be assumed as 45 200 pounds, which, with the weight of track at 235 pounds per linear foot for each girder, makes the dead load 64 000 pounds, and the dead-load shear 32 000 pounds.

The total vertical shear at the support is then 294 100 pounds, and for a specified unit shearing stress of 12 000 pounds per square inch the net section of the web must be

24.51 square inches, upon the assumption that the shear is uniformly distributed over the sectional area. The actual distribution of the shear is illustrated in Art. 59. The minimum thickness of $\frac{3}{8}$ inch allowed by the specification would permit only 18 rivets in the entire depth of 84 inches, and is hence insufficient. A thickness of $\frac{7}{16}$ inch will give a net depth of 56 inches, and if a diameter of 1 inch be deducted for each rivet hole in the web section, it will allow 28 rivets with an average pitch of 3 inches. This thickness will accordingly be used.

The standard rivet in plate-girder construction has a diameter of $\frac{7}{8}$ inch when cold, and the diameter of the rivet holes is made $\frac{1}{16}$ inch larger, so that the heated rivet can be readily inserted. All specifications agree in requiring deductions to be made for rivet holes in tension members with diameters assumed to be $\frac{1}{8}$ inch larger than that of the rivet before driving, but no reference is made to the corresponding deduction in members subject to shear.

It may be added that with the rapidly increasing practice of punching the holes smaller than the rivet diameter and then reaming the holes after the parts are assembled, or of drilling the holes in the solid, the reason which originally led to the deduction of such a large excess for rivet holes is fast disappearing. When these methods of forming the rivet holes are adopted, a clause might well be added in the specification prescribing the use of the actual net section in the computations.

ART. 54. SECTIONAL AREA OF FLANGES.

Since a plate girder under the action of vertical loads is a beam, the fundamental formula for flexure applies to it, namely,

$$M = \frac{SI}{c},$$

in which M is the bending moment at any section due to the

external forces, S the unit stress in the outer fiber whose distance from the neutral surface is c, and I the moment of inertia of the cross-section about the neutral axis.

In order to transform this equation so as to be convenient for the purpose of design, let t be the thickness and h the height or depth of the web plate, while A is the area of cross-section of each flange, and h_1 the distance between the centers of gravity of the flanges, usually called the effective depth of the girder. If the moment of inertia of each flange about its own neutral axis be neglected, as it is relatively small, the following expression may be written for the moment of inertia of the girder:

$$I = 2A\left(\frac{h_1}{2}\right)^2 + \frac{th^3}{12} = \frac{Mc}{S}.$$

Substituting the value of c which is very nearly equal to $\frac{1}{2}h$, and transposing,

$$A = \frac{Mh}{Sh_1^2} - \frac{th^3}{6h_1^2}.$$

But h_1^2/h is approximately equal to h_1, and h^3/h_1^2 is approximately equal to h, whence

$$A = \frac{M}{Sh_1} - \frac{th}{6}; \qquad (1)$$

that is, if the plate girder had solid sections throughout, the area of each flange would be less than that required to resist the entire bending moment, by an amount equal to one-sixth of the section of the web plate.

Although the gross area of the upper half of the girder section may be employed, since it is under compression, only the net section of the lower half should be used. The method therefore adopted is to design the lower flange, which is subject to tension, and then to make the upper flange of the same gross sectional area as the lower one. In applying the

above formula (1), the last term is interpreted as meaning one-sixth of the net section of the web plate. When stiffeners or splices are located at or near the section under consideration, the net section differs considerably from the gross section.

If one-inch rivet holes be deducted for $\frac{7}{8}$-inch rivets, one-sixth of the net section of the web plate becomes 10.3 percent of the gross section when the rivets in the vertical section have the minimum allowable pitch of three diameters or $2\frac{5}{8}$ inches; 11.1 percent for a pitch of 3 inches; 11.9 percent for a pitch of $3\frac{1}{2}$ inches; and 12.5 percent for a pitch of 4 inches, which would rarely be exceeded. If, however, the deduction be made for the actual rivet holes of fifteen-sixteenths of an inch in diameter, the respective percentages will be 11.5 for a pitch of 3 inches, 12.2 for a pitch of $3\frac{1}{2}$ inches, and 12.8 for a pitch of 4 inches.

Some recent specifications state that one-eighth of the gross area of the web plate is to be regarded as effective flange area, but the preceding paragraph shows that this allowance is in many cases somewhat too large. It is not only desirable on theoretical grounds that the resistance of the web plate to flexure should be considered in the design, but also for the practical reason that it encourages the use of thicker web plates and a greater depth where this is not otherwise limited, thereby increasing the life as well as the stiffness of the structure.

Turning now to the design under consideration, the bending moments due to the live load specified in Art. 50 are found at sections five feet apart, according to the method described in Part II, Art. 42. The absolute maximum moment due to this load is found to be 2 703 000 pound-feet at a section 0.2 foot from the center, where the moment is practically the same. The allowance for impact is 1 864 000, and the dead-load

moment is 640 000 pound-feet, making the total bending moment 5 207 000 pound-feet, or 62 484 000 pound-inches.

Placing the backs of the flange angles one-eighth inch beyond the edges of the web plate and assuming the centers of gravity of the flanges to be 1.5 inches less than the distance back to back of angles, the approximate effective depth is $84 + 0.25 - 1.5 = 82.75$ inches. For a specified unit tensile stress of 17 000 pounds per square inch, and assuming 12 percent of the gross web section as effective flange area, the required net area of the lower flange,

$$A = \frac{62\,484\,000}{17\,000 \times 82.75} - \left(0.12 \times \frac{7}{16} \times 84\right) = 44.42 - 4.41 = 40.01,$$

the result being expressed in square inches.

ART. 55. COMPOSITION OF THE FLANGES.

SPECIFICATION. — About one-half of the flange section shall consist of angles, or else the heaviest sections of angles must be used, and the number of cover plates shall be as small as practicable. The cover plates shall be of equal thickness or decrease in thickness outward from the angles, and shall not extend more than four inches or eight times the thickness of the outside plate beyond the outer line of rivets. The net section of the tension flange shall be determined by a plane cutting it square across at any point, and the greatest number of rivet holes which can be cut by any such plane, or whose centers come nearer to it than two and a half inches, are to be deducted from the gross section in computing the net area. The compression flange shall have the same gross section as the tension flange.

The effective diameter of any rivet shall be assumed the same as its diameter before driving; but in making deductions for rivet holes in tension members, the diameter of the holes shall be assumed to be one-eighth of an inch larger than that of the rivet.

One-half of the net flange area, determined in the preceding article, is 20.35 square inches, and hence either $8'' \times 6''$ or $8'' \times 8''$ angles are required. Adopting the latter and observing the rest of the above specifications, the flange may be made up as follows (see Fig. 67):

2 angles, $8'' \times 8'' \times \frac{3}{4}''$; $2(11.44 - 1.50) = 19.88$ square inches.
3 cover plates, $18'' \times \frac{7}{16}''$; $3(7.88 - 0.88) = \underline{21.00}$
Total net section $= 40.88$ square inches.

Two rows of rivets will be required in each leg of the angles, the rivets in adjacent rows being staggered, and hence two rivet holes must be deducted from each angle and from each plate, the pitch of the rivets at the middle of the girder being certainly greater than $2\frac{1}{2}$ inches. For location of rivet lines see Art. 34.

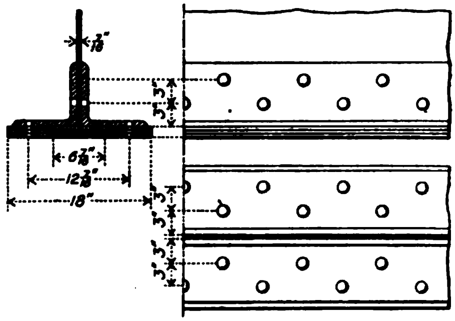


Fig. 67.

The center of gravity of this flange section is next computed and found to be 0.815 inch from the backs of the angles, while the corresponding distance for the gross section of the upper flange is 0.788 inch. The correct effective depth is, therefore, $84 + 0.25 - 1.60 = 82.65$ inches, which makes the revised flange area required equal to $45.15 - 4.41 = 40.74$ square inches. As this value does not exceed the net section given above, the composition of the flanges needs no revision.

If the section were moved so as to cut the adjacent rivets, the distance from the center of gravity of the net section of the lower flange to the backs of the angles would be reduced from 0.815 to 0.705 inch, and the effective depth of the girder increased to 82.76 inches. The average value is used by some designers.

In regard to the deduction of rivet holes for net section, one specification which is extensively adopted provides that the rupture of a riveted tension member is to be considered as

equally probable, either through a transverse line of rivet holes or through a diagonal line of rivet holes where the net section does not exceed by 30 percent the net section along the transverse line.

By comparing the revised and provisional flange areas the student may gain some idea as to the relative effect of small changes in the effective depth or in other items affecting it, and thus learn what degrees of precision are required in the various computations. As the actual sections of shapes are subject to slight variation and there are inaccuracies in workmanship, it is sufficient in practice to determine the effective depth to the nearest tenth of an inch.

While angles can be rolled of any thickness between the minimum and maximum given in the handbooks, the practice is quite extensive to use only standard angles whose thicknesses are expressed in full sixteenths of an inch.

ART. 56. WEB SPLICES.

SPECIFICATION. — Whenever practicable, plate girders shall be built without splices in the web, and when splices are necessary, their number shall be made as small as possible. The splice plates and rivets for the splices shall be such as to develop in every respect the full strength of the net section of the web, the main splice plates extending from flange to flange and having at least two rows of rivets on each side of the joints. In addition to these, two splice plates shall cover the vertical legs of the angles in each flange. The shearing stress on rivets shall not exceed 12 000 pounds per square inch of section, and the pressure upon the bearing surface of the projected semi-intrados (diameter times thickness) of the rivet hole shall not exceed 24 000 pounds per square inch.

According to the Carnegie handbook the extreme length to which a sheared steel plate 84 inches wide and $\frac{7}{16}$ inch thick is rolled is 380 inches. It will thus be possible to build a girder whose span is 80 feet with only two web splices.

In Art. 54 it was shown that one-sixth of the net section of the web is the equivalent flange area regarded as concentrated at the center of gravity of the flange, and which represents the share of the web in resisting the bending moment at any section. The web splice must accordingly be designed to transmit not only the shear in the section but also its proportionate part of the bending moment. This is accomplished with a sufficient degree of precision for all purposes of design when the splice plates and rivets are arranged to develop the full strength of the net section of the web to resist the bending moment only.

As not less than two rows of rivets are to be placed on each side of the joint, the rivets in the lower half of the splice will be arranged as indicated in Fig. 68, the pitch in each row when considered independently being 4 inches toward the neutral surface and 3 inches toward the flange, thus placing the rivets which are the most effective closer together. The numerals on the right of the figure represent the distances in inches from the rivets to the neutral surface.

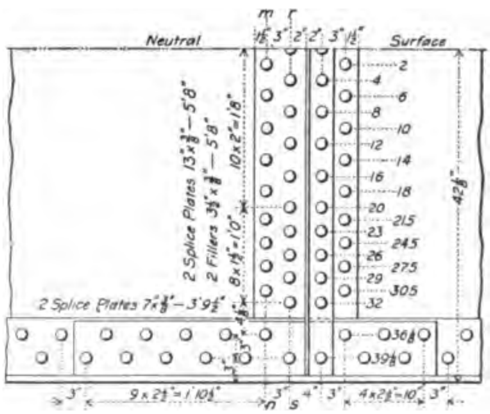


Fig. 68

As the web is $\frac{7}{16}$ inch in thickness and 84 inches deep, and the unit stress in the outer fiber is 17 000 pounds per square inch, the resisting moment of the gross section is

$$\tfrac{1}{6} \times 17\,000 \times \tfrac{7}{16} \times 84 \times 84 = 8\,746\,500 \text{ pound-inches}$$

and that of the lower half is 4 373 300 pound-inches. To find the

resisting moment of the net section of the web it is necessary to deduct that of the diametral sections of the rivet holes in the outer row of the splice. Remembering that for members in tension the diameter of the rivet holes is to be taken as $\frac{1}{8}$ inch greater than that of the rivets before driving, the reduction of tensile stress in the web for a rivet hole at a distance from the neutral surface equal to that of the outer fiber of the web is $17\,000 \times 1 \times \frac{7}{16} = 7440$ pounds, while for one at a distance y from the neutral surface it is $(7440y/42)$ pounds. The moment of this stress about the neutral axis is $7440y^2/42$, and the sum of the moments for all the rivet holes in the row *mn* of Fig. 68 is $7440\,\Sigma y^2/42 = 7440 \times 4714/42 = 835\,100$ pound-inches. This leaves the resisting moment of the net section of the lower half of the web equal to 3 538 200 pound-inches.

With a unit stress of 24 000 pounds per square inch for the bearing on the side of the rivets, the allowable bearing of a $\frac{7}{8}$-inch rivet on the $\frac{7}{16}$-inch web plate is $24\,000 \times \frac{7}{8} \times \frac{7}{16} = 9190$ pounds. The combined strength of the splice plates must equal that of the web, but as no metal less than $\frac{3}{8}$ inch in thickness is allowed in good practice except for filling plates, $\frac{3}{8}$-inch splice plates will be used. Accordingly the bearing of the rivets in both splice plates combined is greater than that in the web. The rivets are in double shear and with a safe unit stress of 12 000 pounds per square inch, the value of a $\frac{7}{8}$-inch rivet in double shear is 14 430 pounds, and hence the bearing in the web governs the determination of the number of rivets in the splice. As the outer row of rivets is $39\frac{1}{8}$ inches from the neutral surface, the bearing value of a rivet at the distance y from the neutral surface is $9190y/39.125$, and the moment of the bearing is $9190y^2/39.125$. The sum of the moments of the bearing values of all the rivets in both rows *mn* and *rs*, exclusive of those in the flange, is

$$9190\,\Sigma y^2/39.125 = 9190 \times 7359/39.125 = 1\,728\,500 \text{ pound-inches.}$$

This result shows that twice as many rows of rivets would be required if no other splice plates were employed except those connecting that portion of the web which lies between the edges of the flange angles. Such an arrangement is not economical, since too large a proportion of the rivets are ineffective in resisting the bending moment. Let two splice plates $7'' \times \frac{3}{8}''$ be placed on the vertical legs of the flange angles, in order to connect those parts of the web plates which carry the highest unit stress. It is now required to find how many rivets, through these plates, are necessary to make the full strength of the splice rivets equal to that of the net section of the web plate. The resisting moment to be taken by the rivets through these longitudinal splice plates is $3\,538\,200 - 1\,728\,500 = 1\,809\,700$ pound-inches, and hence $\Sigma y^2 = 1\,809\,700 \times 39.125/9190 = 7705$ inches^2. Since the squares of 36.125 and 39.125 are 1305 and 1531 respectively, three rivets in each row are required, making $\Sigma y^2 = 8508$ inches^2, that is, six rivets are needed on each side of the joint as shown in Fig. 68. This number of rivets can transmit into the flange angles the full tensile strength of these splice plates, and hence the number of connecting rivets does not need to be increased on that account. However, as the rivets, through the vertical legs of the flanges, have the additional duty to transmit an increment of flange stress from the web plate to the angles, as will be explained in Art. 59, the plates must be extended toward the nearer end of the girder so as to contain enough rivets to take both of these stresses, the pitch being reduced to one-half the value that would otherwise be used. The exact length of the plates will be found in Art. 59.

If the vertical shear be taken into account, the bearing value of the outermost rivet is reduced from 9190 to 8640 pounds, since each rivet must take a shear of 3140 pounds. The allowable tensile stress in the outer fiber is also reduced from 17 000

to 15 800 pounds per square inch, since the total vertical shear at the section (12 feet from the middle) is 126 500 pounds, and the net section of the web is $(84 - 20)\frac{7}{16} = 28$ square inches, the deduction being for 20 rivets, and the resulting unit shear 4520 pounds per square inch. (See Mechanics of Materials, Art. 105.) On introducing these values in the computations, Σy^2 is found to be 7533 instead of 7705 inches^2, thus requiring practically the same number of rivets. The former method, which is much simpler than the latter, is therefore sufficiently precise, as stated above.

The outer row of rivets in Fig. 68 is arranged so as to reduce the net section of the web as little as possible in that part which takes the greatest stress. The resisting moment of the net section of the web is 3 538 200/4 373 300 = 0.809 times that of the gross section, and hence one-sixth of this or 13.5 percent of the gross web area may be regarded as equivalent flange area. This slightly exceeds the value used in Art. 54; viz. 12 percent.

ART. 57. WEB STIFFENERS.

In a plate-girder web which consists only of a continuous web plate there exist at any point compressive and tensile stresses at right angles to each other whose magnitudes equal those of the vertical and horizontal shear at that point (Mechanics of Materials, Art. 143). The lines of the maximum compressive and tensile stresses cross each other at right angles at the neutral surface, and make angles of 45 degrees with that surface. The compressive stresses tend to buckle or wrinkle the web plate, while the tensile stresses tend to keep it straight.

Experience has led to the custom of stiffening the web by means of pairs of vertical angles placed on opposite sides of the web plate, and riveted together, whenever the clear distance between the flange angles is greater than about 50 times the

thickness of the web plate. In the specifications this ratio is given as 50, 60, or 64, or even as high as 80. These stiffeners are usually placed at distances apart not exceeding the depth of the girder, with a maximum limit of 5 or 6 feet, the former value being more generally specified.

Formerly the prevailing practice was to space them closer together toward the ends of the span as the shear increased, but at present the practice of spacing them at uniform intervals is very common. No rational theory has been developed upon which the design of intermediate stiffeners may be based.

It is not definitely known to what extent the addition of intermediate stiffeners modifies the distribution of stresses in the web plate of the girder. Although a few experiments have been made which, together with observations in the maintenance of girders under traffic, throw some light on the subject, it has not received the investigation which its importance seems to demand. Some of these facts are recorded in engineering periodicals, to which references are given at the end of this article.

The intermediate stiffeners should be able to transmit to the web the heaviest concentrated load which may come upon it in a deck girder, or the greatest floor beam reaction in a through girder. In the example used in this chapter the greatest concentrated load is 30 000 pounds, and the addition for impact 24 000 pounds. With the specified compressive unit stress of 17 000 pounds per square inch, a sectional area of 3.18 square inches is required. The leg of the angle which is to be riveted to the web plate must not be less than $3\frac{1}{2}$ inches, as $\frac{7}{8}$-inch rivets should not be used in a smaller size (Art. 34). Two angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, the thickness being the least allowable, furnish much more than the necessary area, but their outstanding legs would not give an adequate support to the 8-inch flange angles which transmit the load to the stiffeners. The size will therefore be increased to $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

The end stiffeners must take the vertical shear from the web and carry it to the bearing plate. They do not act entirely as columns, since the load is distributed along the entire length. A somewhat lower working stress should be taken than that for simple compression,—say about 15 000 pounds per square inch. The sectional area of the end stiffeners must therefore be $294\,100/15\,000 = 19.61$ square inches. Six angles $6'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$ will furnish an area of $6 \times 3.41 = 20.46$ square inches, and may hence be adopted. Sometimes it is specified that the projecting legs of all stiffener angles over the end bearings shall be as wide as the flange angles permit. This rule would accordingly require $7'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$ angles.

All the stiffeners should be closely fitted to the upper flange angles in the deck girders, and the end stiffeners should also be fitted to the lower flange angles to secure a full bearing area. The lower ends of intermediate stiffeners in deck girders, and both of their ends in through girders, require merely a neat fit for the sake of appearance, since they transmit no stresses directly to the flanges. It should be added that many specifications make no distinctions in this respect. The number of rivets connecting the end angles to the web is $294\,100/9190 = 32$, since the bearing value of a $\frac{7}{8}$-inch rivet in a $\frac{7}{16}$-inch web is 9190 pounds. For the sake of uniformity, which simplifies construction, the same number of rivets will be used in all the stiffeners, and equal to that required in the inner rows of the web splice. This will give $3 \times 19 = 57$ rivets, without counting those which also pass through the flange angles, and whose duty is to carry stresses from the web into the flange.

In girders of double-track bridges the reaction of the support may be so large that a sufficient number of rivets cannot be put into the stiffeners. Instead of using wider angles to accommodate two rows of rivets each filler may be widened so as to take an extra row of rivets. The number of rivets then required in

the stiffeners alone is governed by their value in double shear. Sometimes a single plate on each side of the web replaces the separate fillers under the two or more pairs of end angles. See Plate I and Fig. 30.

The ends of plate girders should be finished with cover plates. In deck girders the corners are square, but in through girders the upper corner is generally rounded off to a radius which ranges from one-half to the full length of the bed plate of the support. The upper flange angles are usually cut just before reaching the curve and spliced to angles of reduced thickness which extend around the curve and down the ends, although sometimes the flange angles themselves are extended down to the support (Fig. 46, Art. 44).

The following articles give some idea of the character of the discussions which take place at intervals in regard to the stresses in the webs of plate girders and the function of stiffeners:

Specifications for the Strength of Iron Bridges. By Joseph M. Wilson, and discussion by W. H. Burr and E. Thacher. Transactions American Society Civil Engineers, vol. 15, pages 404, 430, 467, June, 1886.

Vertical or Inclined Stiffeners for Plate Girders. By C. A. P. Turner and J. B. Johnson, Engineering News, vol. 33, page 276, April, 1895. By C. A. P. Turner and J. P. Snow, Engineering News, vol. 33, page 339, May 23, 1895. By Henry Goldmark, Engineering News, vol. 34, page 43, July 18, 1895.

Thermal Condition of Iron and Steel under Stress, and Measurement of Stress by Means of Thermo-electricity. By C. A. P. Turner. Proceedings of the Engineers' Society of Western Pennsylvania, Sept., 1897. This paper contains the results obtained by tests of an experimental girder 10 feet long and $2\frac{1}{2}$ feet deep. For a later valuable paper by Turner, see Transactions of American Society of Civil Engineers, Aug., 1902.

Spacing Stiffeners in Plate Girders. By H. T. Beach, Engineering News, vol. 39, page 322, May 19, 1898. By Practical Bridge Builder, Engineering News, vol. 40, page 10, July 7, 1898. By Joseph M. Wilson and E. Marburg, Engineering News, vol. 40, pages 89 and 90, Aug. 11, 1898. By A. W. Buel, C. A. P. Turner, and Joseph M. Wilson, Engineering News, vol. 40, pages 154 and 155, Sept. 8, 1898. By F. G. Skinner and C. A. P. Turner, Engineering News, vol. 40, pages 339 and 400, Dec. 22, 1898. By H. T. Beach, Engineering News, vol. 41, page 106, Feb. 16, 1899.

Tests of the Stress in Plate-girder Stiffeners. By F. E. Turneaure. Engineering News, vol. 40, page 186, Sept. 23, 1898. This article contains the results of six measurements of the stresses in the stiffeners of a 75-foot plate girder under moving load.

Specifications for Steel Railroad Bridges. Discussion by George S. Morison and J. H. Worcester. Transactions American Society Civil Engineers, vol. 41, pages 184 and 193, June, 1899.

Proposed Specifications for Steel Railway Bridges. By J. W. Schaub, and discussion by H. E. Horton and Ralph Modjeski. Journal Western Society of Engineers, vol. 5, pages 355, 370, and 379, Oct., 1900.

A Direct Method of Spacing Rivets and Finding the Position, etc., of Stiffeners in Plate Girders. By E. Schmitt. With discussion. Transactions American Society Civil Engineers, vol. 45, page 550, June, 1901.

ART. 58. LENGTHS OF COVER PLATES.

In accordance with the requirement of some of the leading specifications one cover plate on each flange will be extended to the end of the girder. The others will be extended at each

end from 9 inches to a foot beyond the point where theory requires them in order to resist the maximum bending moments in the girder.

The combined net area of the two flange angles and one cover plate is 26.88 square inches, while the equivalent flange area of the web is 13.5 percent of its gross section (Art. 56) or 4.96 square inches, making the total flange area 31.84 square inches. The effective depth is found to be 80.9 inches, and hence the bending moment that may be resisted by this section of the girder is $17\,000 \times 31.84 \times 80.9/12 = 3\,649\,000$ pound-feet, and this is the value of the maximum moment at 17 feet from the support. This location is conveniently found by means of a diagram (Fig. 69) whose ordinates represent the maximum bending moments due to the live load, impact allowance, and dead load. Their values, expressed in kip-feet, are given in the following table, one kip being 1000 pounds:

MAXIMUM BENDING MOMENTS.

SECTIONS.	5'	10'	15'	20'	25'	30'	35'	40'
Live load	690	1253	1723	2067	2330	2523	2653	2703
Impact	476	864	1188	1426	1607	1740	1830	1864
Dead load	169	315	439	540	619	675	709	720
Total	1335	2432	3350	4033	4556	4938	5192	5287

The combined net area of the flange angles and two cover plates is 33.88 square inches, which added to the equivalent flange area of the web gives a total of 38.84 square inches. The effective depth is 81.9 inches, and the corresponding bending moment is $17\,000 \times 38.84 \times 81.9/12 = 4\,506\,000$ pound-feet, which is located at $24' \, 6''$ from the support. The approximate length of the outer cover plate will therefore be at least

$2(40' \, 0'' - 24' \, 6'') + 2 \times 9'' = 32$ feet 6 inches, and that of the second cover plate $2(40' \, 0'' - 17' \, 0'') + 2 \times 9'' = 47$ feet 6 inches. The exact lengths of the cover plates will be determined on the drawing after the rivets are located in the flanges, and the flange splices are located, for it is sometimes necessary to extend one cover plate to serve as a splice plate for another one.

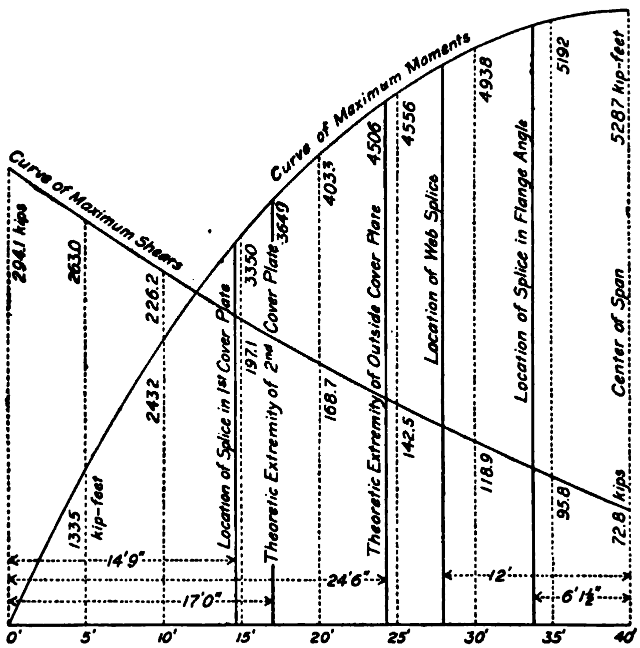


Fig. 69.

When the bending moments are determined by means of an equivalent uniform load, making the moment diagram a parabola, the lengths of cover plates may be quickly found either by the graphic method given by T. K. THOMSON in Engineering News, vol. 32, page 148, Aug. 23, 1894, or by the analytic method given by C. W. HUDSON in the same periodical, vol. 32, page 278, Oct. 4, 1894.

ART. 59. THEORETIC RIVET PITCH IN FLANGES.

The rivets uniting the web plate to the upper flange of a deck girder between any two given sections have two duties to perform: first, to transfer from the flanges to the web whatever load rests directly upon the flanges in this division; and, second, to transmit from the web to the flanges the increment of flange stress developed between these sections. The required number of rivets must then be such as to safely transmit these vertical and horizontal stresses when their resultant is a maximum. The horizontal component of the resultant is considerably greater than the vertical, except near the middle of the span, where in many cases the latter may be even greater than the former.

The maximum difference of flange stress between any two sections occurs when the difference between their respective bending moments is a maximum, provided the effective depth is the same. When the sections are taken a distance apart equal to dx, the difference of moments is dM, and if the entire bending moment were resisted by the flanges, the difference in flange stress would be dM/h_1, in which h_1 denotes the effective depth. The increment of flange stress per linear unit is then dM/h_1dx, which by mechanics equals V/h_1, the vertical shear being designated by V. This difference is a maximum when the maximum values of the vertical shear are inserted in the expression just given. Since the web plate, however, resists a part of the bending moment, V/h_1 must be multiplied by the ratio of the bending moment resisted by the flanges alone to the entire bending moment. This ratio equals that of the area of the flanges to the sum of the flange area and the equivalent flange area of the web as explained in Art. 54. The pitch of the rivets, or their spacing longitudinally, is then obtained on dividing the resistance of one rivet by the value just found. As the rivets connecting the web to the flanges are in double shear, the bearing value of a

rivet on the web plate will usually be less than the double shear, and therefore measures the strength of the rivet to be used in the computation.

The vertical shears whose determination was referred to in Art. 53 are given in the following table and expressed in kips, the sections being taken 5 feet apart. See also Fig. 69.

SECTION =	0'	5'	10'	15'	20'	25'	30'	35'	40'
Live load	155.1	139.9	120.6	104.3	89.6	76.3	64.6	53.3	42.1
Impact	107.0	95.1	81.6	72.8	63.1	54.2	46.3	38.5	30.7
Dead load	32.0	28.0	24.0	20.0	16.0	12.0	8.0	4.0	0.0
Total	294.1	263.0	226.2	197.1	168.7	142.5	118.9	95.8	72.8

The position of the live load which causes the maximum shears is such that the first driver of the locomotive (Art. 50) is just on the right of the section. Its weight on one rail and distributed over three ties, which, with the three spaces, cover 42 inches (Art. 52), is 25 000 pounds. The coefficient of impact for the shears varies from about 0.69 to 0.73, and using the average value 0.71 for this load, the impact allowance is 17 750 pounds. The corresponding weight of the track supported by one girder is 700 pounds, making a total load of 43 450 pounds, or 1035 pounds per linear inch.

At section 0', which is at the support, each flange is composed of two angles and one cover plate (Art. 58), and according to the method given above the increment of flange stress per linear inch resisted by the flange alone is

$$\frac{26.88}{31.84} \cdot \frac{294\,100}{80.9} = 3069 \text{ pounds.}$$

The ratio 26.88/31.84 refers to the tension flange, having been used in Art. 58, but it may be applied to the compression flange as being sufficiently exact for this purpose.

The resultant of 3069 and 1035 pounds is 3236 pounds per linear inch, and the required pitch is $9190/3236 = 2.84$ inches, since the bearing value of a $\frac{7}{8}$-inch rivet in a $\frac{7}{16}$-inch web plate is 9190 pounds.

In a similar manner the pitch is determined at each of the sections, the results laid off as ordinates in Fig. 70, and a curve drawn through their extremities. This diagram gives the theoretic pitch at every point in the half span. The horizontal lines

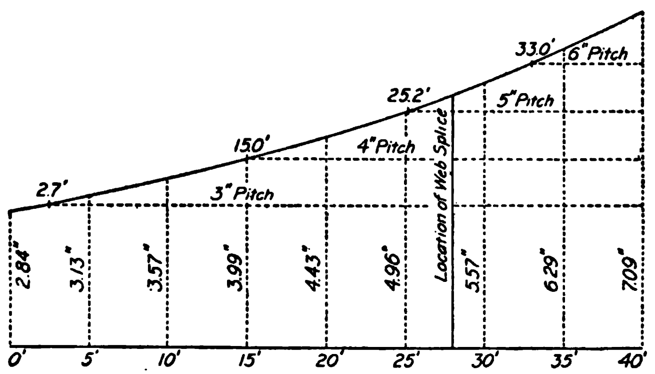


Fig. 70.

show that the pitch is 3 inches at 2.7 feet, 4 inches at 15 feet, 5 inches at 25.2 feet, and 6 inches at 33 feet from the center of the support.

If the vertical component of the rivet stress be neglected, the theoretic rivet pitch in the lower flange is obtained, that at the end of the girder being 2.99 inches, and at the middle 11.69 inches. A comparison of these values with those found for the upper flange indicates the direct influence of the load supported by the flange, on the pitch of the rivets.

The rivets connecting the angles and the cover plates must transmit that portion of the flange stress which is taken by the cover plates. These rivets are in single shear, and hence the strength of a rivet is measured by its value in single shear,

which is less than the bearing in either the cover plate or the angle. The value in single shear of a $\frac{7}{8}$-inch rivet at 12 000 pounds per square inch is 7220 pounds. The proportions of flange stress taken by one, two, and three cover plates respectively are approximately 26, 41, and 51 percent. The rivet pitches at the end of the girder, and at the points where the second and third (or outer) cover plates terminate theoretically (see Fig. 69, Art. 58), are found to be 9.1, 9.1, and 8.9 inches respectively.

For the purpose of comparison the rivet pitch at the end of the girder will also be determined by means of the horizontal shear. The direct effect of the vertical load on the flange rivets will be omitted in this comparison, as it does not affect the result.

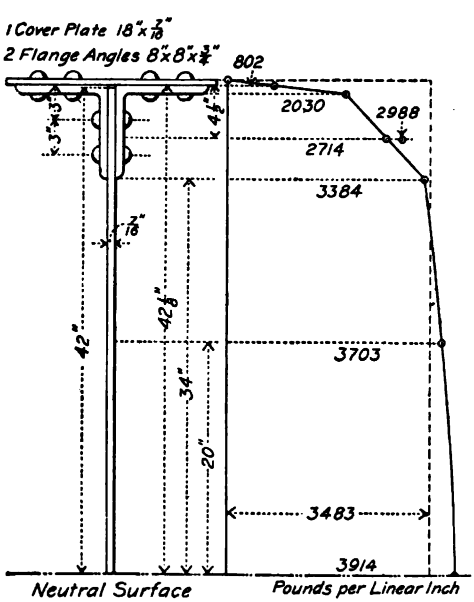


Fig. 71.

On the left of Fig. 71 is shown a vertical section of the upper half of the girder at the support, and on the right is a diagram whose abscissas represent the horizontal shear per linear inch, the values being computed (see Mechanics of Materials, Art. 108) upon the assumption that the cover plate, angles, and web plate are all parts of a single piece of solid metal. This assumed condition is evidently not equal in some respects to that of the girder, in which these parts are riveted together. The horizontal shear in a horizontal

section taken midway between the two lines of the flange rivets is thus found to be 2714 pounds per linear inch.

If the shear between the flange and the web be computed by substituting in the formula the moment of the entire section of both angles as well as of the cover plate, and omitting entirely any part of the web between the angles, the value of the horizontal shear per linear inch is 2988 pounds. This value is observed to be somewhat less than the horizontal shear in the web plate directly adjacent to the flange angles and greater than the shear midway between the rivet lines obtained under the previous assumption, and it probably corresponds more closely to the actual condition of the girder. The resulting pitch is $9190/2988 = 3.08$ inches, while that found by the other method is 2.99 inches, or about three percent less. If, however, in the other method the gross sections be used instead of the net sections, the resulting pitch is 3.03 inches, or only 1.6 percent less, and thereby indicates the magnitude of the error involved in the approximation made in deducing the formula in Art. 54.

The horizontal shear just below the flange angles is 84.5 percent of that at the neutral surface, while the average value of the horizontal shear for the entire section is 3483 pounds per linear inch, or 89 percent of that at the neutral surface. This value is also laid off in the diagram for comparison. Below the flange angles the abscissas represent not only the magnitudes of the horizontal shear per linear inch, but also the equal magnitudes of the vertical shear at the respective distances from the neutral surface. If it be assumed that the entire vertical shear is resisted by the web plate alone, the average vertical shear per linear inch is $294\,100/84 = 3501$ pounds, or 89.5 percent of the shear at the neutral surface. These values show how slight is the error involved in this assumption, and that if a proper allowance be made in adopting the safe unit

stress, the net area of the web section may be found with sufficient accuracy by assuming the shear to be uniformly distributed.

The length of the longitudinal splice plates for the web (Art. 56) may now be determined. The theoretic pitch of the flange rivets at the joint is over 5 inches, while the adopted pitch in this part of the span is 5 inches, and hence the pitch will be reduced near the joint to $2\frac{1}{2}$ inches. If the plates be extended to the left 29 inches, they will contain 11 rivets, $5\frac{1}{2}$ of which are required for the stress due to the web splice, and the remaining $5\frac{1}{2}$ for the increment of flange stress. Theoretically the plates need not extend an equal distance to the right of the joint, or toward the middle of the girder, but only far enough to contain 6 rivets, unless more are required to transmit their stress into the angles by single shear. If 6 rivets are used, they will develop the net strength of the plates. The total length of the plate will therefore be $29 + 16\frac{1}{2} = 45\frac{1}{2}$ inches.

In a plate girder where side plates or vertical flange plates are placed between the angles and the web, all the rivets through the side plates, whether passing also through the angles or not, may be counted in the number necessary to take the horizontal flange increment out of the web plate. To determine the number of rivets which must also pass through the vertical legs of the angles it must be considered that they shall provide sufficient strength in double shear to transmit that portion of the increment of flange stress which is to be taken by the angles and cover plates. The double shear will govern in this case, since its value will be less than their bearing either in both angles or in the combined web and side plates.

For a flange of the composition shown in Fig. 35 the riveting should be so designed as to transfer the increment of flange stress from the web to the several shapes composing the flange in the most direct manner.

A purely graphic method of determining the rivet pitch is described in a paper by E. SCHMITT in the Transactions of the American Society of Civil Engineers, vol. 45, page 550, June, 1901. In the discussion of this paper C. B. WING gives a series of diagrams for the solution of the same problem, covering a wide range of unit stresses and of the other factors involved.

The practical considerations which affect the spacing of the rivets both longitudinally and transversely are given in the next article.

ART. 60. LOCATION OF FLANGE RIVETS.

SPECIFICATION. — The pitch of rivets shall not be less than three diameters when on the same line, nor less than two and one-half times the diameter when staggered. The pitch in the direction of the stress shall never exceed six inches, nor sixteen times the thickness of the thinnest outside plate. When two or more thicknesses of plate are riveted together in compression members, the outer row of rivets shall not be more than four diameters from the side edge of the plate. No rivet-hole center shall be less than one and a half diameters from the edge of a plate, and, whenever practicable, this distance is to be increased to two diameters.

If the theoretic pitch at the end of the girder be less than that required by the specification, the thickness of the web must be increased accordingly. Sometimes three and one-half diameters is specified as the minimum pitch in one line. The smallest angle that will admit two rows of $\frac{7}{8}$-inch rivets is 5 inches (Art. 34), but the rivets in flange angles are usually placed in a single row whenever the vertical leg of the angle is less than 6 inches. While occasionally a single row is used in 6-inch angles, it is expressly forbidden in some specifications. In 8-inch angles it is customary to use two rows, but sometimes three rows are inserted. The location of the pitch lines or rows of rivets is given in Art. 34.

In order to facilitate shop work, the pitch of rivets in deck-plate girders is increased in regular groups from the minimum required at the ends to the maximum near the middle, the

number of changes being few. The object of limiting the maximum pitch according to the specification is to secure a close fit in construction. The pitch is made the same in both upper and lower flanges for economy in manufacture. No change in pitch should be made between two adjacent stiffeners unless necessitated by a flange splice. The spacing in the flange angles must be slightly modified at the location of the stiffeners so as to give sufficient clearance during construction. In through girders the pitch is uniform in each panel.

In the deck girder whose theoretic rivet pitch was determined in the last article the end pitch may be taken as 3 inches, since the pitch was found to be only slightly less in a distance of 2.7 feet from section o′, or the center of the support, while a number of extra flange rivets may be placed in the foot or so which the girder extends beyond that section, and which will more than make up for the difference in pitch. As Fig. 70 shows that the maximum allowable pitch of 6 inches may only be extended to 7 feet on each side of the middle, it is preferable to omit that pitch and employ the 5-inch pitch for 15 feet from the middle. This arrangement will then leave only one intermediate pitch, that of 4 inches.

This specification permits two lines of rivets to be used to connect the cover plates to the angles, when the angles are 6 inches or less in width, but usually requires four lines of rivets when the cover plates are more than about 14 inches wide. If in order to keep down the number of plates it is necessary to widen them beyond the limits already indicated, it is best to increase the width sufficiently to allow a row of rivets on each side outside of the flange angles.

At the ends of cover plates the pitch should be reduced for a short distance so as to reduce the tendency to overstrain the rivets on account of the sudden change in flange section.

A sufficient number of rivets to transmit the full stress for which the plate is designed may be placed at the end of the plate with the smaller pitch. Sometimes this pitch is limited to the minimum used in the flange.

When only one row of rivets is used in each leg of the angles, the horizontal and vertical rivets should stagger, but when two rows of rivets are used in each leg of the angles, the adjacent rows in one angle, whether both are in one leg or not, should stagger. This arrangement places the rivets in the outer row of the horizontal leg of the angle opposite to those of the upper row of the vertical leg. Sometimes those in both rows of one leg are placed opposite points which are intermediate between the adjacent rivets of both rows in the other leg, but this is objectionable on the ground of reducing the net section throughout the span, except near the middle where the pitch is but slightly less, or equal to, the maximum allowed.

In the example given, it was found that the theoretic pitch for the rivets through the cover plates and angles is about 9 inches, which exceeds the maximum allowed. As these rivets must be spaced with regard to those through the vertical legs of the angles, the pitch must either be equal to theirs, or just twice as great, provided the resulting value does not exceed 6 inches. The values to be adopted can therefore be determined in accordance with these statements after the flange and web splices, ends of cover plates, and stiffeners are located on the drawing.

It should be added that a few specifications, including those of WADDELL, direct that the flanges of girders carrying the vertical load from the ties shall have their rivets spaced uniformly from end to end and at the minimum distance employed.

ART. 61. FLANGE SPLICES.

SPECIFICATION. — Splices in flange plates and angles must always be avoided when sufficiently long plates and angles are procurable, which will always be the case unless the span be abnormally long. Where flange splices are un-

avoidable, they must be so located that no two pieces of either the flange or the web shall be spliced within two feet of each other, and so that no flange and web splice shall occur at any point where there is not an excess of sectional area above the theoretical requirements.

The saving in the cost of splices will usually compensate for the extra price which may be demanded for plates and angles of the greatest length obtainable. The object of distributing the splices in the different pieces, and of not allowing any flange splice to come too near a web splice, is to avoid abrupt changes in section which interfere with the proper distribution of stresses in the different members.

As the web splices occur at 12 feet from the middle of the girder, and the outer cover plate extends a few feet farther each way (see Fig. 69, Art. 58), the two angles will be spliced between the center of the girder and the two web splices respectively. The first cover plate will be spliced at two points, so that the second cover plate may be extended sufficiently to serve at each end as a splice plate.

Each of the $8'' \times 8'' \times \frac{3}{4}''$ angles has a net area of $11.44 - 1.50 = 9.94$ square inches, and is to be spliced by an $8'' \times 8''$ angle cut down so as to fit the face of the flange angle, and to have at least the same area. This requires the angle to be seven-eighths of an inch thick, and each leg cut down to about $7\frac{1}{8}$ inches. As the value of a $\frac{7}{8}$-inch rivet in single shear is 7220 pounds, 24 rivets are required to connect the splice angle on each side of the joint. With a pitch of $2\frac{1}{2}$ inches, which is the minimum allowed without reducing the net section of the flange (Art. 55), the length of the splice angle is a little over 5 feet. This would interfere with at least one pair of stiffeners, and hence it is desirable to reduce the length, which can be done by reducing the thickness of the splice angle to nine-sixteenths of an inch, and making up the sectional area by placing a $7'' \times \frac{9}{16}''$ flat on the vertical leg of the opposite flange angle, as shown in Fig. 72.

The flat requires 8 rivets at each end, and the angle 16 rivets, it being remembered that the bearing in the ¾-inch flange angle is greater than the double shear. Accordingly all the rivets in the vertical leg of the splice angle also pass through the flat, and its length is thereby reduced one-third. It should be noticed that these splices are located at points where there is an excess of flange area.

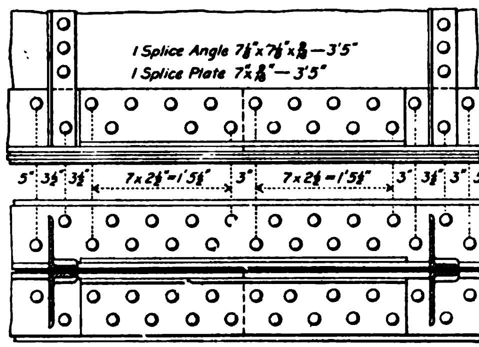


Fig. 72.

Let the nearer flange angle be spliced on the left, and the farther angle on the right of the middle of the girder, the same arrangement being also adopted for the upper flange, and all the splices located, so as not to interfere with any stiffeners.

The net section of the first cover plate is $7.88 - 0.88 = 7$ square inches (Art. 55), and as the second cover plate, which has the same section, is to be extended as a splice plate, single shear will govern the design of the connecting rivets. The number required on each side of the joint is $7 \times 17\,000/7220 = 17$ rivets; but as the rivets are arranged symmetrically in pairs, 18 rivets must be used.

Since the second cover plate is theoretically required at 17 feet from the end (Fig. 69, Art. 58), and the 4-inch theoretic pitch of flange rivets begins at 15 feet (Fig. 70, Art. 59), it is desirable to extend the 3-inch pitch over this splice. The second cover plate will then extend to about 12 feet 6 inches from the center of the support, making its entire length about 55 feet, while that of the middle portion of the first cover plate is 50.5

feet. These lengths are subject to a slight modification on account of the location of the stiffeners, which interfere somewhat with the regularity of the rivet pitch.

This extension of the 3-inch rivet pitch, and the use of a $2\frac{1}{2}$-inch pitch for about 2 feet at the end of the outer cover plate leaves less than 7 feet remaining for the 4-inch pitch. It should therefore be considered whether it may not be better to omit that pitch altogether.

WADDELL's specification requires that every non-continuous flange piece shall be fully spliced so that the splicing plates and rivets shall have a calculated strength at least 25 percent greater than that of the net section spliced. Under this requirement the computation given above would have to be changed accordingly.

ART. 62. LATERAL BRACING.

SPECIFICATION. — The lateral bracing shall be proportioned for a static wind load of 150 pounds per linear foot on each system. The system connected to the loaded flanges shall be proportioned also for a moving wind load of 300 pounds per linear foot. The compression flanges of the girder shall be so stiffened laterally that the unsupported length shall not exceed 12 times the width of flange. All members shall be so proportioned that the tensile unit stress shall not exceed 17 000 pounds per square inch, nor the compressive unit stress to exceed 17 000 pounds per square inch reduced in proportion to the ratio of the length to the least radius of gyration of the section, by the following formula: $p = 17\,000/\left(1 + \frac{1}{11\,000} \cdot \frac{l^2}{r^2}\right)$, in which p is the permissible working stress per square inch in compression, l the length of piece in inches between centers of connections, and r the least radius of gyration of the section in inches. No compression member in the wind bracing shall have a length exceeding 120 times its least radius of gyration. For members of any importance, more than two rivets are to be used for each connection. For unit stresses on rivets, see Art. 56. In field riveting the number of rivets found by the specified unit stresses shall be increased 25 percent if driven by hand, or 10 percent if satisfactory power riveters are used.

In the specifications which refer to the spacing of cross-frames, its value ranges from 12 to 20 feet. In order to make them equidistant in an 80-foot span, it is necessary either to space them 20 feet, 16 feet, or 13 feet 4 inches. Adopting the intermediate distance the panel length of the upper lateral system becomes also 16 feet for a Warren type of bracing, and may be reduced to 8 feet by adding substruts at every panel point. Every alternate substrut is a member of the cross-frame or transverse bracing. The upper lateral system holds in line the compression flange of each girder. As the cover plates are 18 inches wide, the allowable unsupported length is 18 feet. The substruts which form no part of a cross-frame are therefore not required on this account, but may be inserted in accordance with the best practice. They will be omitted in the lower system. The skeleton diagrams of the upper and lower lateral systems are shown in Figs. 73 and 74 respectively.

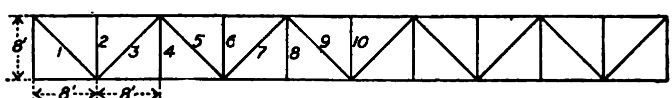


Fig. 73.

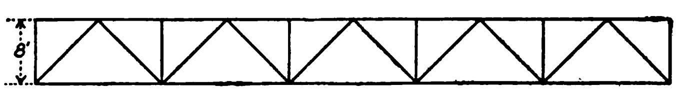


Fig. 74.

For the specified wind loads the maximum stresses in the diagonals are as follows: $S_1 = \pm 22\,900$, $S_3 = \pm 18\,200$, $S_5 = \pm 13\,800$, $S_7 = \pm 9700$, and $S_9 = \pm 5900$ pounds. As the double signs are due to the wind blowing in opposite directions, and the reversals of stress do not take place in rapid succession as in stresses due to live load, it is customary not to design the lateral system for alternate stresses.

Since the ratio of the length l of a diagonal to its least radius of gyration r is to be limited to 120, the least allowable radius

of gyration is $106/120 = 0.88$ inch, and a reference to one of the handbooks shows that for a single angle no smaller size than $6'' \times 4''$ or $5'' \times 5''$ can be used. For the former size, in which l/r is 120, the specified column formula gives an average compression per square inch of 7360 pounds, and hence the required area of the end diagonal is $22\,900/7360 = 3.11$ square inches. The area of a $6'' \times 4'' \times \frac{3}{8}''$ angle is found to be 3.61 square inches, while r is 0.88 inch, the value assumed. The thickness of this angle is the least allowed, but on account of the eccentric end connections of the angle its sectional area will probably have little to spare.

Let an investigation be made to see whether the sum of the stresses in the outer fiber, due to both the column action and the eccentric connection, falls within the allowable limit of 17 000 pounds per square inch. First, let the angles be riveted to the connecting plates by the 6-inch leg, and let bending in a vertical plane be considered. According to the handbook, the distance from its center of gravity to the back of the longer flange is 0.94 inch, its moment of inertia I about the neutral axis parallel to the longer flange is 4.90 inches^4, and the corresponding value of the radius of gyration r is 1.17 inches. The angle tends to bend so that the concave side is on the back of the longer flange, and since $l/r = 106/1.17 = 91$, the maximum compressive stress on that side is

$$S' = \frac{22\,900}{3.61}\left(1 + \frac{91 \times 91}{11\,000}\right) = 11\,100 \text{ pounds per square inch.}$$

The bending moment due to the eccentric connection is $22\,900 \times 0.94 = 21\,500$ pound-inches, and by means of the formula deduced in Mechanics of Materials, Art. 102, the compressive stress in the outer fiber on the same side of the angle which is due to this moment is

$$S'' = \frac{21\,500 \times 0.94}{4.90 - \dfrac{22\,900 \times 106 \times 106}{9.6 \times 29\,000\,000}} = 5100 \text{ pounds per square inch;}$$

in which 29 000 000 is the coefficient of elasticity. The total stress is therefore 11 100 + 5100 = 16 200 pounds per square inch.

A similar computation for the second case, when the angles are connected by the 4-inch leg, gives 8800 + 6900 = 15 700 pounds per square inch. If, in the first case, the bending moment due to its own weight be included, the unit stress is increased 200 pounds per square inch. Repeating the computation for an angle $5'' \times 5'' \times \frac{3}{8}''$, the result is 9000 + 5700 = 14 700 pounds per square inch. The area of both angles is the same, and a comparison of the maximum unit stresses indicates their relative strength.

By connecting the other leg of the angle to the connecting plate by means of an angle clip, the eccentricity which tends to produce bending in a horizontal plane may be eliminated, but that for bending in a vertical plane still remains, since the entire stress is transmitted by the rivets through the connecting plate. Since the eccentricity is very small in the plane in which the angle tends to bend as a column, — that is, in a plane perpendicular to the neutral axis, with respect to which the radius of gyration is a minimum, — this investigation does not need to be carried further. It may be noted that the net section of one leg of the angle is sufficient to transmit the tension in the end diagonal, as this requirement is sometimes specified when only one leg of the angle is attached. Although the stresses in the remaining diagonals are less than in those at the end of the span, the same size is required throughout, since l/r is limited to 120.

The length of the lateral braces which are perpendicular to the girders is 82 inches, and hence the radius of gyration must

not be less than $82/120 = 0.68$ inch. As the stress is only 1800 pounds, a $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle will have abundant strength. No narrower angle than $3\frac{1}{2}$ inches will admit a $\frac{7}{8}$-inch rivet, according to the standard given in Art. 34. The conditions already referred to require the same sized laterals to be used in the lower system as in the upper one, although the stresses are less than one-third as large.

Since the connecting rivets are in single shear, their shearing value will govern, and as the rivets through the laterals are field and not shop rivets, their number must be increased 25 percent for hand riveting. These conditions require 4 rivets in the connection of the end diagonal, but an additional rivet is needed on account of its eccentricity. This result may be tested as follows: The longitudinal shear in each rivet is $22\,900/5 = 4580$ pounds. If three rivets be placed in the pitch line which is $1\frac{3}{4}$ inches from the back of the 5-inch angle and two rivets in the line which is 2 inches from the other one, the center of gravity of the shearing surfaces of the rivets is $1.75 + 0.8 = 2.55$ inches from the back of the angle, or 1.16 inches farther from it than the center of gravity of the angle. The moment of rotation in the plane of the shearing surfaces, caused by the stress in the angle, is therefore $22\,900 \times 1.16 = 26\,570$ pound-inches, and this moment produces a shear in each rivet whose value is directly proportional to its lever arm. Two of the rivets are 5.1, two are 2.8, and one is 0.8 inches from the center of rotation, and if the shear in the most distant rivet is P, the moment of the shear in all the rivets is

$$(\overline{5.1}^2 + \overline{5.1}^2 + \overline{2.8}^2 + \overline{2.8}^2 + \overline{0.8}^2)\ P/5.1 \text{ pound-inches.}$$

Equating this to the moment of rotation and solving for P, its value is found to be 1980 pounds. The direction of this shear is perpendicular to the lever arm of 5.1 inches, while that of the shear of 4580 pounds is parallel to the axis of the angle. Their

resultant is found graphically to be 5280 pounds. The allowable stress for field rivets is 20 percent less than for shop rivets, or 5770 pounds. Five rivets are therefore required. The moment of rotation in a vertical plane tends to produce tension in some of the rivets, but as the connecting plate bends easily in that direction the rivet tension must be small. No less than three rivets should be used in connecting any lateral, even though that number is not theoretically needed.

Although the laterals are usually designed to take the wind stress only, it should be remembered that their principal duty is to resist the lateral vibrations caused by the live load passing over the bridge at full speed. In view of the great increase in live loads, it is a question whether the assumption that these stresses do not exceed those computed for the wind pressure leads to a sufficient provision for lateral stiffness. When it is considered that these vibrations cause rapid reversals of stress, a material increase in lateral stiffness would be secured by treating the wind stresses as live-load stresses, and designing the laterals for alternate stresses.

In designing members for alternate or reversed stresses, one of the best specifications is to find separately the areas required for both tension and compression, and to add three-fourths of the smaller area to the larger one in order to obtain the total sectional area of the member. The rivets, however, are to be computed for the sum of the two stresses. On applying this to the lateral system under consideration, the end diagonal of the upper system must be increased to $\frac{1}{2}$ inch in thickness, the rest remaining unchanged. If clips be used so as to reduce the eccentricity of the connections, the numbers of field rivets required in the diagonals of the upper system are respectively 8, 7, 5, 4, and 3.

In some cases it may be advisable to go a step further and increase the stresses by allowance for impact. Practically the

same result may, however, be secured by increasing the prescribed wind pressure. The Atchison, Topeka, and Santa Fé Railway uses a moving wind load of 500 pounds per linear foot and an equal static wind load in its standard designs for plate-girder bridges.

ART. 63. TRANSVERSE BRACING.

The form and composition of the transverse bracing were described in Art. 43. The object of the intermediate cross-frames is to increase the general stiffness of the bridge, and all the angles composing its horizontal and diagonal braces will be taken as $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ for the reasons indicated in the previous article. Each connection should have three rivets, and where the diagonals cross they should be riveted to a small connecting plate.

The end cross-frame must transfer the reaction of the upper lateral system to the support. This reaction is 18 000 pounds. If it be assumed that one-half of the reaction is transferred to the support by each diagonal, the areas required in all the members will be less than those of the small angles already adopted for the intermediate cross-frames. As it is very important that the end bracing shall be rigid, it is best to use the larger shapes which are employed in the best practice. Each diagonal may be composed of one angle $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, the upper horizontal of two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, and the lower horizontal of two angles $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. (See also Plate I, Art. 69, and the references in Art. 49.)

ART. 64. BEARINGS AT SUPPORTS.

The principal types of expansion bearings now in use were described in Art. 44. As indicated in that article there are considerable differences in the specifications with regard to the length of span below which sliding bearings and above which

roller bearings are to be used respectively. Most specifications give this span as 75 or 80 feet. A few require rollers when the span exceeds 60 feet, while others permit sliding up to 85 or 90 feet. Several specifications require some form of rocker bearing or hinge joint for spans from 50 feet to 65 or 70 feet, and in exceptional cases even beyond this limit.

Hinge bolsters should be used in combination with rollers in order to secure a uniform distribution of the load on the rollers with whatever deflection the girder may sustain under its live load.

The expansion end of the girder shall be free to move longitudinally for a variation in temperature of 150 degrees Fahrenheit, but must be anchored against lifting or moving sideways.

When cast-steel shoes are employed, as in Figs. 51 and 52, their design must make the following provisions: Adequate bearing area of the vertical ribs on the pin; a pin of sufficient diameter to resist the bending moment produced on account of the outer ribs of one shoe being farther apart than those of the other shoe; vertical longitudinal ribs of the necessary strength as double cantilevers to carry the load from the stiffeners at the ends of the upper shoe to the pin, and in the lower shoe to distribute the pin reaction as a uniform load to the rollers; and vertical transverse ribs and bearing plates of ample thickness to make the distribution of pressure uniform transversely. These ribs have the additional duty of stiffening the longitudinal ribs and aiding them to resist any transverse horizontal thrust that may be brought upon them in service. If the greatest allowable pressure in pounds per linear inch is specified as $p = 600\,d$, in which d is the diameter of the rollers, their aggregate length is found on dividing the gross reaction of the girder including the shoes by this allowable bearing. The bearing area of the bed plate under the rails must be such as not to exceed a safe

value for the material composing the bridge seat. Where impact is taken into account, this value may be taken as about 400 pounds per square inch. WADDELL specifies permissible pressures for ten different materials.

The anchor bolts at the fixed end must be designed to take the combined shear and tension due to the tendency for the cast-iron bolsters to slide and overturn when the brakes are applied to the train crossing the bridge at full speed. The horizontal tractive load thus applied to the girders is to be taken as 20 per-cent of the greatest live load that can be placed on the bridge.

When bolsters are built up of plates and shapes, the same general method of design is followed. Whether two or three vertical plates shall be used depends upon the size of the girder. Transverse webs should be employed so as to secure the proper distribution of loading in that direction, unless this can readily be done by bearing plates of moderate thickness without too great a stress in flexure. The vertical legs of the connecting angles should be wide enough to allow two rows of rivets. It is often specified that no bearing plate, bed plate, vertical plate, or connecting angle should be less than three-quarters of an inch in thickness, and sometimes the minimum for the bed plate is made seven-eighths of an inch. No rollers less than 3 inches in diameter are allowed, while the best practice makes the minimum diameter 4 inches.

The segmental rollers with parallel sides shown in Fig. 48, Art. 44, are the standard adopted by the bridge department of the New York Central and Hudson River Railroad.

For additional information relating to the design of segmental rollers see Art. 81. References to the bearing power of friction rollers are given in Art. 98. Formulas for the investigation and design of cylindrical rollers are deduced in Mechanics of Materials, Art. 156.

ART. 65. ESTIMATE OF WEIGHT.

The following weights are computed with the aid of the tables in a handbook:

MATERIAL FOR ONE-HALF OF THE GIRDER.

Flanges:

4 angles, $8'' \times 8'' \times \frac{3}{4}'' \times 40'\ 10''$, @ 38.9 lbs.	6354 pounds.	
2 cover plates, $18'' \times \frac{7}{16}'' \times 40'\ 10''$, 2 cover plates, $18'' \times \frac{7}{16}'' \times 27'\ 6''$, 2 cover plates, $18'' \times \frac{7}{16}'' \times 16'\ 3''$, } @ 26.79 lbs.	4532	10 866

Flange splices:

2 cover angles, $7\frac{1}{8}'' \times 7\frac{1}{8}'' \times \frac{9}{16}'' \times 3'\ 5''$, @ 26.6 lbs.	182	
2 plates, $7'' \times \frac{9}{16}'' \times 3'\ 5''$, @ 13.39 lbs.	91	273

Web:

1 plate, $84'' \times \frac{7}{16}'' \times 28'\ 9\frac{3}{4}''$, $\frac{1}{2}$ plate, $84'' \times \frac{7}{16}'' \times 23'\ 11\frac{3}{4}''$, } @ 124.96 lbs.	5099	5 099

Web splice:

2 plates, $13'' \times \frac{3}{8}'' \times 5'\ 8''$, @ 16.58 lbs.	188	
4 plates, $7'' \times \frac{3}{8}'' \times 3'\ 8\frac{1}{2}''$, @ 8.93 lbs.	132	320

Stiffeners:

24 angles, $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 6'\ 10\frac{1}{2}''$, @ 11.7 lbs.	1931	
1 angle, $7'' \times 3\frac{1}{2}'' \times \frac{7}{16}'' \times 6'\ 10\frac{1}{2}''$, @ 15 lbs.	103	
$6\frac{1}{2}$ fillers, $3\frac{1}{2}'' \times \frac{3}{4}'' \times 5'\ 8''$, @ 8.93 lbs.	329	
2 plates, $20'' \times \frac{3}{4}'' \times 5'\ 8''$, @ 51 lbs.	578	2 941
1 end cover plate, $18'' \times \frac{3}{8}'' \times 7'\ 4''$, @ 22.96 lbs.	168	168
Total		19 687

ONE-HALF OF UPPER LATERAL SYSTEM.

Braces: 1 angle, $5'' \times 5'' \times \frac{1}{2}'' \times 9'\ 4''$, @ 16.2 lbs.	151
4 angles, $5'' \times 5'' \times \frac{3}{8}'' \times 9'\ 4''$, @ 12.3 lbs.	459
$2\frac{1}{2}$ angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 6'\ 10''$, @ 8.5 lbs.	145
10 connecting angles, $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, aggregating $6'\ 2''$, @ 10.4 lbs.	64
16 connecting $\frac{3}{8}''$ plates, aggregating 22.56 sq. ft., @ 15.3 lbs.	345
	1 164

ONE-HALF OF LOWER LATERAL SYSTEM.

Braces: 5 angles, 5″ × 5″ × ⅜″ × 9′ 4″, @ 12.3 lbs.	674
4 connecting angles, 5″ × 3½″ × ⅜″, aggregating 2′ 4″, @ 10.4 lbs.	24
8½ connecting ⅜″ plates, aggregating 13.72 sq. ft., @ 15.3 lbs.	210
	808

END CROSS-FRAME.

2 angles, 5″ × 3½″ × ½″ × 8′ 3″, @ 13.6 lbs.	224
2 angles, 3½″ × 3½″ × ⅜″ × 6′ 7½″, @ 8.5 lbs.	113
2 angles, 5″ × 3½″ × ⅜″ × 6′ 7½″, @ 10.4 lbs.	138
4 connecting angles, 7″ × 3½″ × $\frac{7}{16}$″ × 1′ 8¾″, @ 15 lbs.	105
5 connecting ⅜″ plates, aggregating 9 sq. ft., at 15.3 lbs.	138
6 ⅜″ washers	5
	423

INTERMEDIATE CROSS-FRAME.

2 angles, 3½″ × 3½″ × ⅜″ × 8′ 10″, @ 8.5 lbs.	150
2 angles, 3½″ × 3½″ × ⅜″ × 6′ 9″, @ 8.5 lbs.	115
5 connecting ⅜″ plates, aggregating 4.7 sq. ft., @ 15.3 lbs.	72
	337

DEAD LOAD FOR ONE GIRDER, EXCLUDING TRACK.

1 girder, 2 × 19 486 lbs.	39 374 pounds.
½ of upper lateral system	1 164
½ of lower lateral system	808
1 end cross-frame	723
2 intermediate cross-frames	674
3036 pairs of rivet heads, @ 0.452 lb.	1 372
Gross weight for a length of 81′ 8″	44 115 pounds.
Net weight for a length of 80′ (the span)	43 215 pounds.

The net weight was assumed to be 45 200 pounds, and the difference of 1985 pounds is found to be less than one percent of the sum of the equivalent live load and the actual dead load. The stresses will therefore not require revision.

The following table gives the weights of the various parts of the structure, exclusive of track and pedestals, and the corresponding percentages of the entire weight:

	Weight in Pounds.	Percentage of Total Weight.	
Flanges	21 772	49.4	50.6
Flange splices	546	1.2	
Web	10 198	23.1	38.7
Web splices	640	1.5	
Stiffeners and end cover plates	6 218	14.1	
Half upper lateral system	1 164	2.6	7.6
Half lower lateral system	808	1.8	
Cross-frames	1 397	3.2	
Rivets	1 372	3.1	3.1
Total	44 115	100.0	100.0

As the weight of the girder depends not only upon the given loads, but also on the unit stresses and many other details prescribed by the specifications, it is difficult to deduce a general formula for the weight. The above analysis, however, makes it possible to estimate the total weight very closely at an early stage of the design, for the combined weight of the flanges and web plate is 73.2 percent of the entire weight of the girder and bracing. This percentage has but a small range for different specifications in combination with a large range of live load.

At first, let the weight per linear foot be assumed as six to seven times the span in feet, the larger value being used for the heaviest live loads, and then let the web section and the composition of the flange be designed in accordance with the specifications adopted. The approximate lengths of the cover plates may be quickly found by dividing the maximum ordinate of the moment diagram in proportion to the respective flange areas, and locating the corresponding ordinates. If the weight of these items be increased by about one-third, the result will differ but little from the final estimate of the weight.

ART. 66. ECONOMIC DEPTH.

It is of interest to observe what absolute as well as relative variations in the weight will be obtained by changing the depth of the girder. The weight of the flanges varies inversely as the effective depth, while that of the web, together with its splices and stiffeners, varies nearly as the depth of the web plate for relatively small changes in depth, and these two depths differ only by amounts ranging from 1.35 inches at the center to 2.1 inches at the end of the span in this example. Slight changes from the economic depth do not appreciably affect the weight of the girder, hence these variations in depth should produce about equal changes in the weights of the flanges and of the web with their respective details. The minimum material results when these weights are about equal, as was shown in Art. 11.

Experience shows that in order to compute the corresponding weights for different depths the results will usually be sufficiently close by assuming the weights of the flanges to vary inversely as the depths of the web plate. For a depth of 96 inches the weight of the flanges and their splices will be about $(21\,772 + 546)84/96 = 19\,530$ pounds, while the weight of the web and its details will be about $(10\,198 + 640 + 6218)96/84 = 19\,490$ pounds. This shows that 96 inches, or one-tenth of the span, is the depth which requires the minimum material. The reduction in the total weight is only about 350 pounds, and the percentages are 44.6 and 44.6 instead of 50.6 and 38.7 given in the table in the preceding article. Considerations relating to the cost of manufacture and the limitations imposed by required clearances and the grade line of the railroad, generally make the true economic depth somewhat less than that which gives the minimum weight.

ART. 67. CAMBER.

The extensive adoption of plate girders for increasing spans in recent years has led to the practice of providing a camber so

that when the bridge is loaded the track will not sink below the horizontal. According to the standards of the Northern Pacific Railway, for a span of about 100 feet the web plates are spliced so as to give a camber of one inch before the girders are erected, and afterwards the cross-ties are notched so as to leave a camber of $\frac{1}{2}$ inch in the finished unloaded bridge. For a span of 80 feet these amounts are reduced 25 percent.

In the track elevation in Chicago, camber was provided in some girders of still shorter spans, five-eighths of an inch being put into girders of 68 feet span, one-half of which remained in the unloaded spans when completed. In some others of the same span the initial camber was $1\frac{1}{4}$ inches and the final camber $\frac{1}{2}$ inch. Some specifications, however, prescribe that plate girders shall have no camber.

ART. 68. DETAIL DRAWINGS.

Instead of publishing the general plans of the girder whose design, with the exception of a few minor features, is given in this chapter, there are shown on Plates I and II in the next article the full detail drawings of a girder bridge of the same span, this being one of the standard plans of the Northern Pacific Railroad. These plates are published by the kind permission of H. E. STEVENS, Bridge Engineer. The plans are practically shop drawings, as the full dimensions are given for every piece and all the members and rivets are located.

The student should examine Plate I carefully and note the differences between the details shown and those designed in this chapter. Special attention is called to the wooden floor and its connections; to the full-length flange angles; to the character and positions of the web splices; to the modification of the rivet spacing in the flanges on account of the stiffeners and splices; to the intersection of the rivet lines of the lateral

angles in the web plate of each girder; to the composition of the diagonals in the end cross frame; and to the connection by both legs of the horizontal transverse braces to the girders.

The details of the end bearings, whose complete shop drawings are given on Plate II, require no additional explanation. The care with which every detail has been designed is manifest. The form of the segmental rollers and of the roller plate is the Morison standard, which is described in Art. 81.

The student will also find it advantageous to make a comparative study of the details of the plate-girder bridges, to which references are given in Art. 49.

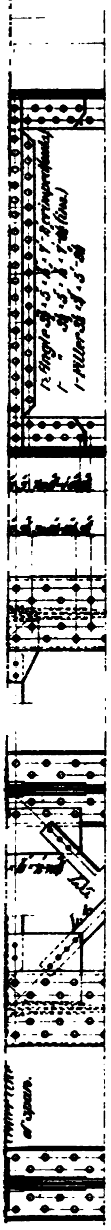
ART. 69. STANDARD PLANS.

The modern movement in American practice to standardize the details of construction has been extended by some railroads to the design of plate-girder bridges. Among these may be mentioned the Northern Pacific, the Harriman Lines, the Santa Fé System, and the Great Northern railroads.

The standard plans of the Northern Pacific Railway include complete drawings of deck plate-girder bridges from 30 to 100 feet in span, and of through bridges from 40 to 100 feet, both kinds varying by 5 feet in span. In the Journal of the Western Society of Engineers, vol. 6, page 51, Feb. 1901, may be found a paper on Northern Pacific Standard Bridge Plans by RALPH MODJESKI, who prepared them as consulting engineer. It is illustrated by reproductions of a number of plans and a diagram of weights. Since this paper was written a part of the plans have been redesigned to provide for a heavier loading. One of these revised plans is shown on Plates I and II.

On the Atchison, Topeka and Santa Fé Railway the standard plate-girder bridges are divided into four classes. Class A includes deck girders from 26 to $105\frac{1}{2}$ feet in length, out to out,

R-10-1018
COMPLETE SET
R-10-1018
R-10-1018
R-10-278
Rev. as to guard rail 7-7-02



BILL OF TRACK MATERIAL
FOR ONE SPAN

Material.	No. Pcs.	Size	Length	Mark
Cross Ties.	40	13"·9"	12'-0"	
" "	32	13"·9"	13'-0"	
Guard Timber.	10	6"·8"	16'-6"	
Floor Bolts	50	3/4	1'-8"	R ◻
Boat Spikes	84	1/2	0'-10"	
◻ See standard drawing R.10 278.				

Estimated weight exclusive of bearings, 95,740 lbs.

Camber. Web plates to be spliced so as to produce a camber of 3/4 at center of span. Cross ties to be framed so as to leave a camber of 3/8 in finished unloaded span.

Notes.

Material. All material (except when specified otherwise) to be medium steel. Rivets and bolts to be soft steel.

Reaming. Holes to be punched with a 3/4 dia. die and reamed to 7/8 dia. after assembling. Rivets to be 7/8 dia.

Stiffeners. All stiffeners to have a driving fit between flange angles.

Painting. Surfaces in contact to be given one heavy coat of red lead and boiled linseed oil before assembling. Surfaces inaccessible after erection to be given two coats of the same paint. All other surfaces to be given one coat of boiled linseed oil.

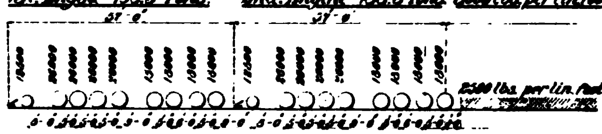
General. Span calculated for standard loading.

Max. Bending Mom. from R.L.+U.L.	3021900	ft. lbs.
" " " " D.L.	660000	"
" End Shear " R.L.+U.L.	173300	"
" " " " D.L.	33000	"
Extreme fiber stress per sq. in. bottom flange,	11100	"

In size of angles, first figure indicates leg of angle shown.

Diagram showing standard loading for moments and shears.

1st. Engine 133.5 Tons. 2nd. Engine 133.5 Tons. 5000 lbs. per lin. foot.



N. P. R'Y.
STANDARD PLAN.
80 FT. DECK PLATE GIRDER.

SCALE 1/4 - 1'

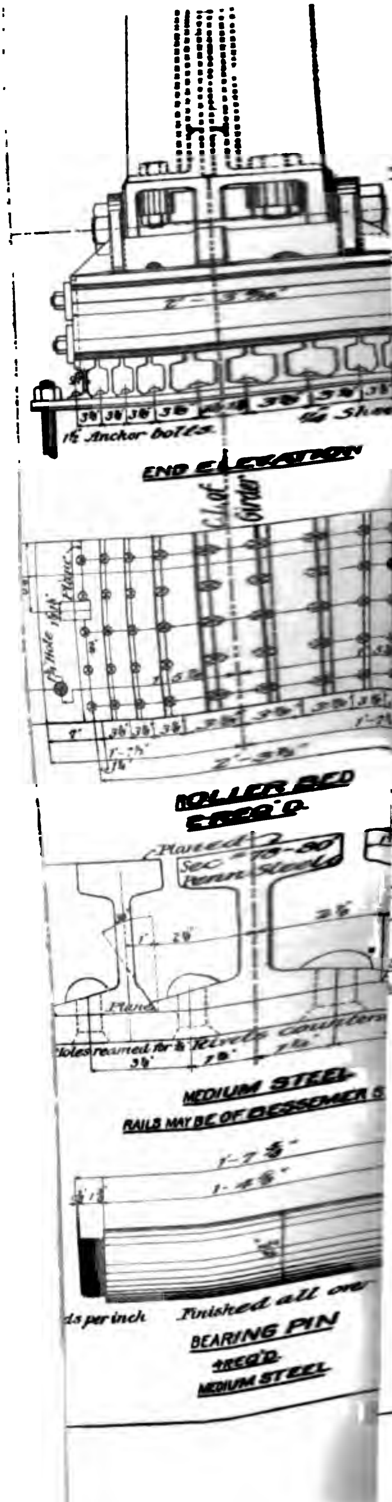
Approved ....

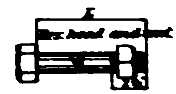
Chief Engineer.

RALPH MODJESKI
CIV. ENGINEER.
Monadnock Bldg.
CHICAGO.
Oct. 1901.

LITHO. BY CHAS. HART 36 VESEY ST. N.Y.

R-10-





BILL OF BOLTS

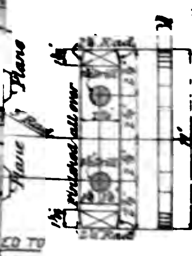
No.		d	L
[illegible]	Turned Bolts	[illegible]	[illegible]
4	" "	[illegible]	5"
*4	[illegible] Anchor Bolts	[illegible]	[illegible]
* See Sheet R-10-[illegible]			

Notes

Each set of Expansion Bearings must be fitted together complete at the shop and ma

All Pins and Rollers to be finished with a water cut.

Estimated weight of one set of bearings.



N. P. R'Y.

STANDARD DETAILS

OF

END BEARINGS.

70 TO 85 FT INCL. DECK PLATE GIRDERS.

70 TO 85 FT INCL. THROUGH PLATE GIRDERS

SCALE, 1½", 3" & 6" = 1

Approved ....

Chief Engineer.

Revised as to Lower Shoes June [illegible] 1906

LITHO. BY CHAS. HART 36 VESEY ST.

designed for economy of weight. The depth ranges from a little over one-seventh to a little less than one-tenth of the effective span. The cross-frames are only from 8 to 10 feet apart and divide the lateral systems into panels, intersecting diagonals being employed in each panel of both systems.

Class B consists of deck girders from 26 to 85 feet in length, which are intended for locations where it is necessary to make them as shallow as the limits of deflection permit. For lengths from about 45 to 85 feet the general arrangement is about the same as for class A, except that the depth is reduced to one-thirteenth or one-fourteenth of the span. The flanges have vertical side plates to avoid too many cover plates and to accommodate the larger number of rivets needed to connect the flanges to the web plates. For lengths from 26 to 42 feet, four lines of girders are used, and they are so spaced that each track rail is midway between a pair of girders. The depth varies from 2′3″ to 2′8½″, which ranges from one-eleventh to one-fifteenth of the effective span. There is no lateral system in this case, but the four girders are connected by cross-frames with solid webs at intervals ranging from about 6 to 11½ feet. These bridges are proved by experience to have unusual lateral stiffness.

Class C includes through girders from 60 to 105½ feet in length, designed with long panels so as to economize material in the floor system. The girders have the same depths as in class A. The panels vary in length from about 14½ to 17 feet, and the floor beams are about 3½ feet deep. In the long spans four lines of stringers are used in order to reduce the economic depth of the floor. The spacing of the girders on tangents is 17 feet 2 inches.

Class D consists of through girders from 26 to 105½ feet in length, in which the floor system is designed as shallow as possible without reference to economy in weight. The panel lengths

vary from about 8 to 12 feet, and four lines of stringers are used for all spans.

Class B is not employed for lengths exceeding 75 feet, as the saving in depth would not warrant it, class C or D being substituted for it under these conditions. Classes A, B, and D have lengths increasing by increments of 3 to 5 feet, and in classes C and D additional plans are made adapted to curves of 5 and 10 degrees. The weights of the bridges increase in the order of the class letters for any given span, the shipping weights for a span of 60 feet, for example, comparing as the percentages 100, 121, 156, and 176. No expansion rollers are used in any case, but rockers are employed at one end in spans exceeding 75 feet in length.

An excellent article giving a more detailed description of these standards may be found in Engineering News, vol. 49, page 482, May 28, 1903. It contains a table of the estimated weights of plate-girder bridges for classes A, B, C, and D, and for spans from 26 to 105½ feet. See also the article in Engineering Record, vol. 48, page 598, Nov. 14, 1903.

Eight standard detail plans of deck plate-girder bridges on the Harriman Lines are published in the Railroad Gazette, 1905, vol. 38, pages 248, 278, 310, 328, 347, 370, and 389. The lengths vary from 30 to 100 feet and in all cases no cover plates are used on the upper flanges.

CHAPTER VIII.

DETAILS OF RAILROAD PIN BRIDGES.

ART. 70. FORMS OF TRUSSES.

A comparison of the leading bridge specifications and railroad standards indicates that the preferred lower limit of span for plate girders ranges from 19 to 30 feet, that for riveted trusses from 100 to 110 feet, and that for pin-connected trusses from 150 to 250 feet. In 1911 there is a decided tendency among railroad bridge engineers to raise the lower limit of spans for pin-connected truss bridges.

The riveted trusses are most frequently made either of the Warren type or of the Warren with sub-verticals, the Pratt truss being employed to some extent for the longer spans. The New York Central and Hudson River Railroad introduced in 1899 riveted trusses of the Baltimore type for spans from 100 to 200 feet, which prior to that time had been applied only to pin-connected trusses and to spans exceeding the larger limit named. Some details of riveted trusses are given in Chapter XI.

The Pratt is the prevailing type for the shorter spans of steel pin-connected trusses. The Warren truss with sub-verticals has been used in a few cases like that on the terminal improvements at Providence, R. I. (see Railroad Gazette, vol. 27, page 457, July 12, 1899), and that on the terminal improvements at Richmond, Va. (see Engineering News, vol. 44, page 379, Nov. 29, 1900). Formerly Warren pin trusses were employed more frequently, but it appeared later as though they would go out

of use entirely. Pegram trusses are used to a very limited extent on the Union Pacific and several other western railroads.

As the span increases the Parker truss is usually employed, while for still larger spans the Baltimore and Pennsylvania trusses, with their subdivided panels, are successively adopted. The ranges of span for the Pratt, Parker, Baltimore, and Pennsylvania trusses in actual use overlap one another to a remarkable degree. It is practically impossible to account for the wide variation in practice regarding the adoption of different types of trusses. In 1911 their upper limits of span were 287, 407, 517½, and 668 feet respectively. See Art. 71 in Part I.

ART. 71. OPEN FLOOR AND STRINGERS.

In through bridges there are generally two stringers to a track spaced from 6½ to 8 feet apart, which support the track ties. The details of the ties, guard rails, etc., are about the same as for deck plate-girder bridges, except that alternate ties are frequently extended the full width for a footwalk. A few railroads, like the Boston and Maine, use four lines of stringers under each track, the main stringers being placed directly under the track rails, while the safety stringers are about 2½ feet outside of the others. The uniformity of the spacing of the crossties is broken by the floor beams, which support the stringers; but as the top flange of the floor beams is seldom more than a few inches below the tops of the ties, a derailed wheel will pass over the wider space in safety.

In some deck bridges of short span the ties are extended over the full width of the bridge and rest upon the chords of the trusses, as in the case of deck plate girders. As the span increases and with it the spacing of the trusses, this type of floor increases in cost and deflection, and is replaced by one of the same kind as that used for through bridges. In this case

the upper chords of the trusses frequently act also as safety stringers. See the report on bridge floors, to which reference was made in Art. 45.

When the panels are very short, the stringers may consist of I-beams, but generally their construction is similar to that of plate girders of short span. The flanges either consist of two angles or of two angles with one cover plate. The practice of not allowing cover plates is becoming quite prevalent, since it affords a better bearing for the ties, and simplifies the work of track maintenance. In some cases the web is extended $\frac{1}{2}$ or $\frac{3}{4}$ inch above the flange angles, thus obviating the necessity of notching the ties for the full width of the flange.

The stringers of each track are united by a lateral system of the Warren type attached to the upper flanges and by an intermediate cross-frame. Both of these features are used in long panels, and only one of them in short panels, some engineers using the lateral system in this case, while others use the cross-frame only. A cross-frame is also inserted at the ends of end stringers when there is no floor beam at the end of the bridge. The elevation of an intermediate stringer and of part of an end stringer, together with that of a cross or sway frame, is shown on Plate III. It will be noticed that there are no intermediate stiffeners in this example.

While the lateral system of stringers is generally of the simple Warren type, sub-struts are occasionally employed at the other panel points, as well as where the cross-frames are placed. On the inset of Engineering News, Jan. 11, 1900, may be seen an example where a double intersection Warren bracing is used. This arrangement, however, is quite unusual.

In through bridges the ends of the stringers are usually riveted to the webs of the floor beams between their flange angles by means of pairs of connecting angles and of bracket

angles, as indicated on Plates III and IV, Art. 82. Sometimes, however, the upper flange angles are extended over the floor beam, and the web cut out so as to clear the flange of the floor beam. (See Fig. 75.) Splice plates connect the tops of the adjacent stringer flanges, thus making them practically continuous and relieving the upper rivets in the connecting angles from tension when the adjoining panels are loaded. This arrangement also permits the ties to be spaced uniformly.

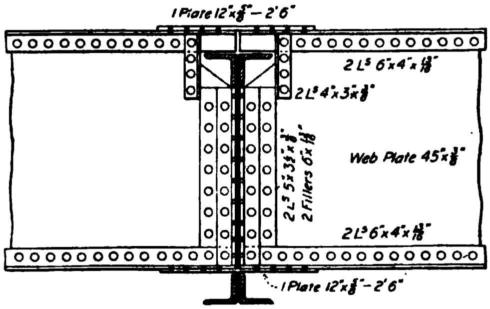


Fig. 75.

In deck bridges the stringers frequently rest on top of the floor beams, as illustrated on Plate V. The lateral system of the bridge may then be connected to the bottom of the stringers, the top of the floor beams, and the bottom of the chords of the trusses, and thereby avoid bending the floor beam horizontally by the tractive force developed on applying the brakes to the train.

When, however, the stringers are connected to the webs of the floor beams and the lateral system is connected to the top flanges of both floor beams and stringers, the web of the stringer may be extended far enough above the regular flange so as to attach secondary flange angles, on which to receive the ties. The projecting web and secondary flange are cut to allow the laterals to pass. This arrangement was adopted in the New Glasgow bridge, whose characteristic details are shown in Engineering Record, vol. 43, page 241, March 16, 1901.

The longest stringers in any simple truss bridge in America are in the Municipal bridge over the Mississippi river at St. Louis, erected in 1911, their span being 48 feet.

ART. 72. SOLID FLOORS.

Several types of the trough floors described in Art. 46 are used in pin-connected truss bridges as well as in girder bridges. Some of the references given in Art. 47 contain descriptions and illustrations of their details when so applied. In an article on the Willamette bridge at Portland, Ore., in Railroad Gazette, vol. 21, page 260, April 19, 1889, the drawings show a splayed-channel trough floor riveted to the sides of the stiff lower chord of the trusses. In Engineering News, vol. 36, page 406, Dec. 17, 1896, may be seen the application of a trough system like Fig. 64, Art. 46, to the floor under the double-track railroad of the double-deck highway and railroad bridge at Rock Island, Ill. The floor is laid upon four lines of stringers, and continuous plates, $20'' \times \frac{3}{8}''$, are placed under the rails and riveted to the troughs so as to form an effective lateral bracing.

In the 348-foot span of the Victoria bridge at Montreal, the double tracks are laid on a continuous half-inch floor plate which is supported by transverse 24-inch I-beams spaced only about 14 inches apart. These I-beams are connected to the webs of longitudinal plate girders lying in the planes of the trusses and riveted to the posts below the lower chords. Longitudinal plates, $10'' \times \frac{1}{2}''$, are riveted on top of the floor plates under each rail.

The inset of Engineering News, Aug. 24, 1899, shows the plan of a solid floor built up of 12-inch channels and plates on the upper deck of the Wells Street bridge in Chicago. Two channels with their webs vertical, their flanges toward each other, and their backs $11\frac{3}{4}$ inches apart are connected by a top flange plate. Similar pairs of channels and cover plates are spaced 12 inches apart in the clear and connected by 12-inch channels with their webs horizontal and their backs at about the

mid-height of the vertical channels. The ties for the tracks of the Northwestern Elevated Railroad were laid directly in the shallow troughs of this floor without any ballast.

ART. 73. FLOOR BEAMS.

The floor beams of truss bridges are similar to those of through plate-girder bridges. The objection to cover plates in the case of stringers does not hold for floor beams. In through trusses a part of the web has to be cut away in order to clear the lower chord, and in order to secure enough space for rivets in the end

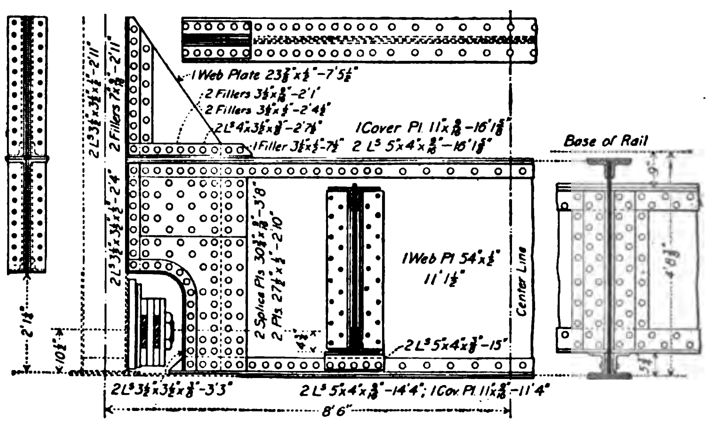


Fig. 76.

connecting angles the web must be extended beyond the upper flange. One form of construction is shown in Fig. 76. The web is spliced so that the end plate may extend up the required distance. The splice plates are continued to the end so as to act also as filler plates and to aid in strengthening the web around the cut. The connecting angles pass over the vertical legs of the upper flange angles, neither of the legs being cut away, and the curved angles pass over the lower end of the connecting angles. An additional pair of filler plates is there-

fore required under the curved angles, and they are extended beyond the angle to give increased strength and to simplify the construction. The lower flange and the bottom of the post are connected by a plate to which the diagonals of the lateral system are also attached.

In Fig. 77 is shown the end of a floor beam in the Pratt truss whose side elevation is given in Fig. 111. The inner edge of the extended web plate is stiffened by a pair of small angles. Several other special features will be noticed, especially the stiffeners between the stringer connections. Another form for a through bridge is shown on Plate III. Sometimes the lower flange angles are bent up outside of the stringers to take the place of the separate inclined angles, in which case another pair of short horizontal angles is riveted to the bottom of the web plate as illustrated clearly in the Engineering Record, vol. 43, page 244, March 16, 1901. When the floor beam is not extended down past the lower chord, the eccentric connections of the lateral system cause a bending moment in the bottom of the post which is avoided in the forms just described.

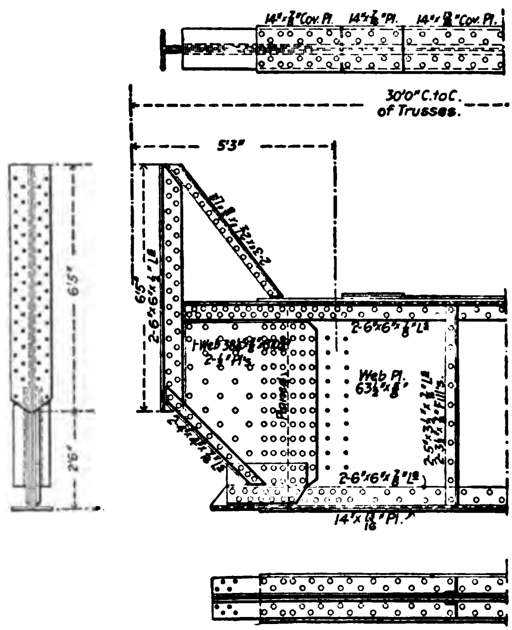


Fig. 77.

In the Port Perry bridge over the Monongahela river this result is secured in another way. A trapezoidal web plate stiffened with angles is riveted to the bottom of the floor beam just inside of the lower chord and also to the horizontal connecting plate of the lateral system which is attached to the bottom of the post. The effect of this construction is to cause a negative bending moment in the floor beam which neutralizes a part of the positive bending moments due to the dead and live loads. (See Fig. 78.)

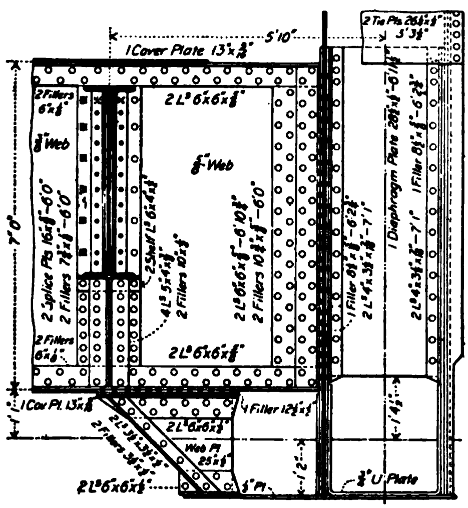


Fig. 78.

The floor beam of a deck bridge is shown on Plate V. The upper corner of the web plate is cut away to clear the diagonal eye-bars of the truss. The stiffeners below the stringers are required to distribute the concentrated loads to the web of the floor beam, fillers being put under the angles. An example in which the top of the floor beam is level with the top of the upper chord is given in Engineering Record, vol. 41, page 126, Feb. 10, 1900.

In double-track bridges the floor-beam flanges may be increased by means of side plates, as in plate girders. In the Bellefontaine, the Alton, and the Delaware river bridges this arrangement is adopted for the upper flanges only, while in the Rankin bridge it is adopted for both flanges. See Engineering Record, vol. 44, page 467, Nov. 16, 1901.

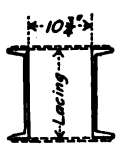
When a floor beam is not riveted to a post, but to some plates or to a short member which resembles a post in construction, but connects with a tension member, like the sub-vertical in a Baltimore or in a Pennsylvania truss, or the suspender of a Pratt truss, as in Fig. 111, Art. 82, the floor beam is effectually stayed against rotation by rods extending to the adjacent panel points. The connection of a floor beam with the extension of a post below the lower chord is illustrated in Railroad Gazette, vol. 25, page 651, Sept. 1, 1893.

In all of these examples a diaphragm is required in order to carry its share of the load from the floor beam to the outer half of the post. It consists of a web plate united by a pair of angles to the two sides of the post.

Not many years ago end floor beams were employed in only a few cases, and those in trusses of large span. Now they are frequently used in short spans as well, and a number of railroads have adopted them as the standard construction.

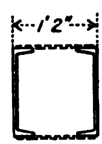
ART. 74. INTERMEDIATE POSTS.

The simplest form of post consists of two channels, whose flanges are united by short plates at or near the ends, called tie plates, and by lattice bars between. When the flanges are



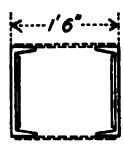
2-15" [s 33 lbs.

Fig. 79.



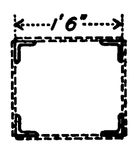
2-15" [s 33 lbs.

Fig. 80.



2-15" [s 48.5 lbs.
2 Pls. 12"×½"

Fig. 81.

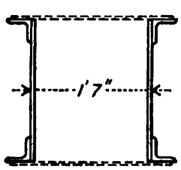


4 Ls 3½"×3½"×½"

Fig. 82.

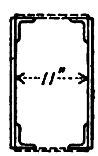
turned out (Fig. 79) as in the older practice, it is necessary to cut the channel flanges near the joints, as indicated in Fig. 111, Art. 82. When the flanges are turned in, as in Fig. 80, this cutting may be avoided and a stronger column secured for the

same out-to-out measurements. When the largest channels do not furnish sufficient area, the section is sometimes increased by adding two plates, preferably on the inside, as in Fig. 81. When still larger sections are required, the post is built up with plates and angles, as shown in Fig. 83. This form is sometimes said to consist of built channels. In Fig. 84 the angles are turned in, the advantage of so doing being the same as for rolled channels. The increasing area required for the posts toward the end of the span is obtained by increasing the thickness of the parts, or in case the thickness becomes excessive, by



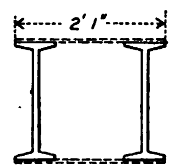
4 L^s 4"x4"x$\frac{11}{16}$"
2 Pls. 24"x$\frac{5}{8}$"

Fig. 83.



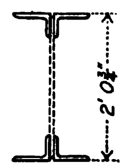
4 L^s 4"x3"x$\frac{5}{8}$"
2 Pls. 22"x$\frac{11}{16}$"

Fig. 84.



2-20" I^s 90 lbs.

Fig. 85.



4 L^s 6"x4"x$\frac{3}{8}$"

Fig. 86.

adding an additional plate on each side, either of the full width of the side plates or to fill only the clear width between the angles.

The posts of the Victoria Jubilee bridge at Montreal have an unusual composition. Two I-beams are laced together (see Fig. 85) for each of the posts, 20-inch and 18-inch I-beams being employed in the posts near the ends and middle of a span respectively.

Fig. 84 shows how relatively narrow a post is sometimes made so as to be packed with the connecting diagonals in the upper chord. See Engineering Record, vol. 41, page 126, Feb. 10, 1900. On the other hand a post like Fig. 83, whose plates are only 22 inches wide, has the backs of the angles spaced 31¼ inches, in order to enter the outer spaces of the upper chord with its f-ur webs. See Engineering Record, vol. 41,

page 516, June 2, 1900. Figs. 82 and 86 show additional post sections, which are mainly used for the sub-verticals of Baltimore and Pennsylvania trusses, which support the upper chord midway between the long posts. The former section has also been used for collision struts.

Elevations of intermediate posts showing the tie plates and lattice bars which connect the two halves of the posts, as well as their diaphragms opposite the floor-beam connections, may be seen on Plates III, IV, and V, and in Fig. 111.

ART. 75. MAIN AND COUNTER DIAGONALS.

The simplest form used for a main tie consists of one or more pairs of eye-bars (Plate III). Tables of the standard sizes of eye-bars may be found in all of the handbooks. Sometimes, in order to secure stiffness in the panels of short spans requiring no counter bracing, the eye-bars are connected by riveting an angle to each bar and uniting the angles with lattice bars. In the panels which require counterbracing the same result is secured by using two pairs of angles laced together to form an I-section. (See Fig. 86 and Plate III.) In members with larger sectional areas a solid web plate is substituted for the lacing.

When the main ties are eye-bars the counters in the same panel consist either of an adjustable eye-bar, or of a square bar with loop eyes, when the required section is small. When laced angles are used for the main ties, the counters have the same composition.

Another method of securing greater stiffness has been adopted to some extent in which the counter ties are omitted and the main diagonals designed to take both tension and compression. The member is then made up either of two rolled channels laced together or of built-up channels, each one being composed of a web plate and two angles. The bridge over the Missouri river

at Bellefontaine, Mo., may be mentioned as a prominent example in which counterbraced diagonals are used, whose composition is the one mentioned last.

The larger vibration due to adjustable counters and the great difficulty in keeping them in proper adjustment has led to the design of the other forms, and so far as they have been compared under traffic, there is little or no difference between the action of Pratt trusses having counterbraced diagonals which take both tension and compression and those in which both main and counter ties are riveted members.

ART. 76. SUSPENDERS.

In the through Pratt truss the suspender or hip-vertical is the vertical tie which connects the upper end of the inclined end post and the second panel point of the lower chord. In the Baltimore and Pennsylvania trusses there are not only the long suspenders, but a number of short ones whose duties are similar. These members have all the forms of section which were mentioned for the diagonals, whether counterbraced or not. If eye-bars are used, they are frequently connected by bent bars instead of by angles and the ordinary forms of lattice bars. (See Fig. 111, Art. 82.) When channels are employed, the flanges may either be turned in or out, and the same is true when the channel section is built up. The sectional area of built-up channels is increased sometimes by using double webs. When the I-section is used in a large truss, two flange plates are added to the two pairs of angles. For examples of the forms mentioned see Plates III and V, and Engineering Record, vol. 41, page 516, June 2, 1900, and vol. 37, page 384, April 2, 1898.

As the suspender in a through Pratt truss receives its stress only from loads in the first two panels, its stress changes more rapidly than that of any other member, and it also receives its

impact more directly. In order to reduce the excessive vibration thus produced some railroads require the suspender to be made of a riveted post section in all cases. This arrangement also prevents rising driftwood from buckling the floor and pulling the bridge off the pier.

In a deck Pratt truss with inclined end posts the only duty of the suspender is to support the lower chord members, and hence in this case it is made of a square bar with either upset or loop-welded eyes, or of two angles laced together so as to form a member about as wide transversely as the intermediate posts. The stiff member is preferable.

ART. 77. LOWER CHORD MEMBERS.

In simple pin-connected steel trusses the lower chord members are very seldom made of anything else than eye-bars, except in the two panels at each end. The depth of eye-bars used in trusses of ordinary span generally does not exceed 8 inches. On the other hand, the smaller depths are not now used to such a great extent as formerly, since it is considered desirable to use few comparatively heavy bars rather than a larger number of light ones. (See Plate IV.)

The largest eye-bars that have been used in any simple truss bridge in this country are those of the Municipal bridge over the Mississippi river at St. Louis, built in 1911, their depth being 16 inches, the greatest thickness $2\frac{1}{16}$ inches, and with panel lengths of 30, 38, 45, and 48 feet. Eye-bars 10 inches deep are used in the Louisville, Bellefontaine, Alton, and Rankin bridges, the greatest thickness being respectively $2\frac{1}{2}$, $2\frac{5}{8}$, $2\frac{1}{16}$, and $2\frac{1}{8}$ inches. In the Bellefontaine bridge the bars extend over two panels of the Baltimore trusses, being 55 feet long between centers of pins. In Fig. 111 are shown two pairs of eye-bars $51' \ 3\frac{3}{4}''$ long, the inner ones being riveted to the

suspender and the outer ones resting on the horizontal legs of a pair of connecting angles.

In the best practice the lower chord members in the first two panels at each end of the span are designed to resist both ten-

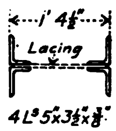


Fig. 87.

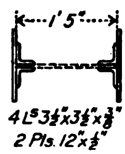


Fig. 88.

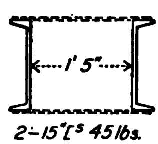


Fig. 89.

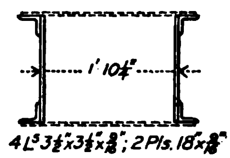


Fig. 90.

sion and compression. This construction enables the lower chord to resist the compression caused by the traction load when the brakes are applied to the train, or the thrust of a derailed car on the bridge, or that caused by a derailed car striking the end of the truss. It also reduces vibration, and increases the stiffness of the truss, especially in short spans. The principal forms of section are shown in Figs. 87 to 93 inclusive. Fig. 91 gives the section used in the end panels of the Alton bridge, and Fig. 92 those in the Bellefontaine bridge.

In a few cases the lower chord of pin-connected trusses is constructed with plates and angles from end to end. Fig. 93

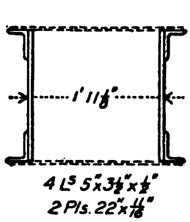


Fig. 91.

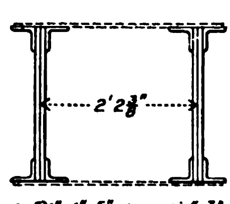


Fig. 92.

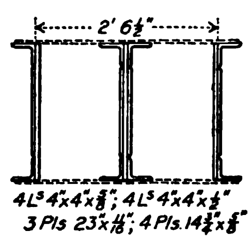
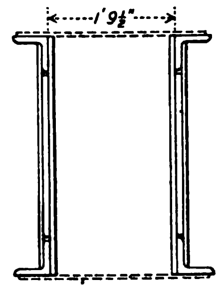


Fig. 93.

gives the section in a panel toward the middle of one of the fixed spans of the United States bridge at Rock Island. In the end panels of the bridge only two webs are employed. Fig. 94

gives the section of the lower chord of the International bridge at Buffalo. The chord is made very deep in order to resist the flexure caused by the floor beams, which are spaced only half the distance between the panel points of the trusses. This construction was used to secure a shallow floor. The floor beams consist of 24-inch I-beams, and the stringers of 4 lines of 15-inch I-beams. See Engineering Record, vol. 43, page 567, June 15, 1901.



4 Ls 6"×6"×⅞"; 2 Pls. 40"×⅞"
2 Pls. 27½"×⅞"; 2 Pls. 38"×7/16"

Fig. 94.

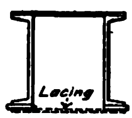
The bridge department of the Baltimore and Ohio Railroad has designed some spans in the vicinity of 150 feet in which the use of eye-bars is restricted to the end ties and the entire bottom chord, all the bars being laced together in order to eliminate as far as possible the vibration of these members. Sometimes the eye-bars in the end panels only are laced instead of using members composed of plates and shapes, as shown in Fig. 111, Art. 82. The use of bottom chords which are stiff throughout is also referred to in Chapter XI.

ART. 78. UPPER CHORD AND END POSTS.

One of the simplest sections of an upper chord member is shown in Fig. 95. The flats below the channels are used to balance the section about a horizontal axis passing through the centers of the channel webs. These are often omitted, but unbalanced sections are not regarded favorably by the best designers. When the section is so small that the required thickness of the metal is less than the minimum allowed, the cover plate and flats are omitted and then the top of the member is laced as well as the bottom.

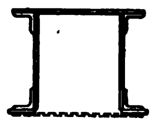
The compositions indicated in the two examples given in Figs 96 and 97 are much more frequently employed for ordi-

nary spans. In the one case the section is balanced by means of flats, while in the other the lower angles are increased in size for the same purpose. The former method is preferred, as it simplifies the construction at the joints where pin plates



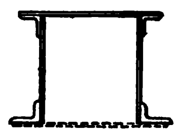
1 Cover Plate 19" × 3/8"
2–15" [s 33 lbs.
2 Flats 4" × 3/4"

Fig. 95.



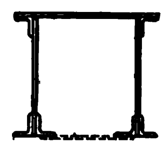
1 Cover Pl. 22" × 3/8"
2 Web Pls. 16" × 3/8"
4 Ls 3½" × 3½" × 3/8"
2 Flats 4½" × 3/4"

Fig. 96.



1 Cover Plate 24" × 7/16"
2 Web Plates 18" × 1/2"
2 Angles 3½" × 3½" × 3/8"
2 Angles 5" × 3½" × 7/8"

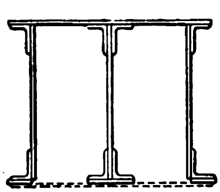
Fig. 97.



1 Cover Plate 24" × 7/16"
2 Web Plates 20" × 5/8"
4 Angles 3½" × 3" × 7/16"
2 Ls (inside) 3½" × 3½" × 1/2"

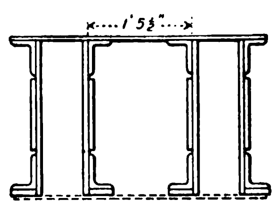
Fig. 98.

must be attached to the sides in order to secure sufficient bearing on the pins. In Fig. 98 the section is balanced by using two angles instead of one at the bottom of each web plate. At the panel points the horizontal legs of the inner angles are cut to afford the necessary clearance for the posts and diagonals. The latticing is connected to the inner angles only. This section is taken from the Northern Pacific Railway's standard plan for a 200-foot through pin bridge dated Oct. 5, 1899.



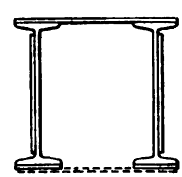
1 Cover Plate 34" × 3/8"
2 Web Plates 26" × 9/16"
1 Web Plate 26" × 3/4"
8 Angles 5" × 3½" × 7/16"
2 Flats 4" × 5/8"
1 Flat 8" × 5/8"

Fig. 99.



1 Cover Plate 43" × 1/2"
2 Web Plates (inside) 25¼" × 7/8"
2 Web Plates (outside) 25¼" × 3/4"
4 Plates 12" × 3/4"
8 Angles 6" × 4" × 3/4"
4 Flats 4" × 1"

Fig. 100.



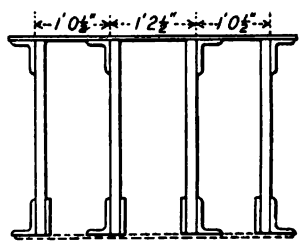
1 Cover Plate 27½" × 5/8"
2–24" I-Beams 100 lbs
2 Web Plates 20 × 5/8"
2 Flats 7½" × 5/8"

Fig. 101.

Additional area is obtained not only by increasing the thickness of the plates and shapes, but also by putting additional

web plates in the clear space between the angles or by placing a web plate of the full depth inside of each of the others. Fig. 99 shows a section containing three webs, and in this case also the outer webs are strengthened in the manner just described. The maximum upper chord section of the Bellefontaine bridge is given in Fig. 100. That of the Delaware river bridge is similar to this except that the inner upper angles are placed on the outside of the inner webs as indicated on Plate V, which shows some details of another bridge on the same division of the Pennsylvania Railroad.

Fig. 102 gives the composition of the largest section of the upper chord of the Monongahela river bridge at Rankin, Pa., its sectional area being 334.52 square inches. It is the largest chord section of any simple truss in use (1902). It will be noticed that the flats are placed opposite the vertical legs of the angles instead of being riveted to their horizontal legs. The chords of the heavy truss in the Monongahela river bridge at Port Perry, Pa., are a little wider, but the depth and area are less. The composition is as follows: 1 cover plate, 50″ × $\frac{5}{8}$″; 2 pairs of outer web plates, 30″ × $\frac{13}{16}$″; 2 pairs of inner web plates, 30″ × $\frac{5}{8}$″; 4 upper angles, 4″ × 4″ × $\frac{5}{8}$″; 2 outer lower angles, 6″ × 4″ × $\frac{3}{4}$″; 2 inner lower angles, 6″ × 6″ × $\frac{7}{8}$″; 2 outer flats, 6″ × $\frac{1}{2}$″; and 2 pairs of inner flats, 6″ × $\frac{5}{8}$″. The arrangement of the shapes is similar to that in Fig. 102, except that the outer flats are placed between the outer angles and the web plates. Five intermediate lines of rivets, with a large pitch, are used to connect the several pairs of web plates. The light truss in the same bridge has only three webs. In both bridges the



1 Cover Plate 48″ × $\frac{5}{8}$″
4 Web Plates 33″ × 1¾″
8 Angles 6″ × 4″ × ¾″
4 Flats 6″ × ¾″

Fig. 102.

ends of the chords and end posts where pin bearing is required have short angles placed opposite the upper angles and extended the full length of the pin plates.

The new trusses of the International bridge at Buffalo, erected in 1901, have upper chords of a very unusual section, shown in Fig. 101. Toward the ends of the span the side plates are reduced, and finally omitted. The lacing at the bottom consists of $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ angles. At the panel points portions of the inner flanges of the I-beams are cut away to provide the clearance needed to pack the web members.

When the chords and end posts have either three or four webs, it is important that their ends be prevented from shifting their relative position after the pin holes are bored, or else trouble is caused in erection. The same conditions apply to the sections where the chords are spliced. This is accomplished by means of transverse diaphragms, as indicated on Plate V. It will be noticed that between the inner and outer webs the diaphragms consist simply of two angles, while between the inner webs plates are also used.

The construction of end posts is usually the same as that of the chords in the same span, the variations rarely being more than those between the upper chord members in different panels. Occasionally the width of the end post may be different from that of the upper chord, but this is rather exceptional.

ART. 79. LATERAL BRACING.

Formerly the upper lateral ties of through bridges consisted of adjustable square bars or round rods connected either to the top of the upper chord or to the middle of its inner web by means of connecting plates and pins. In long spans two sets of ties were often used connected to the top and bottom of the chord respectively. This construction is now seldom employed,

nearly all the standard specifications for railroad bridges stating that stiff members are preferred for the lateral bracing. Those who still use the adjustable members claim that they are not only much lighter, but that the upper chord can be more thoroughly lined up by this means. The object of the stiff laterals is to secure greater lateral stiffness in the bridge, as well as to avoid the difficulty of maintaining the rods in proper adjustment. Many specifications state that it is preferable to avoid altogether the use of adjustable members in trusses, lateral and sway bracing.

Stiff lateral diagonals are most frequently composed of single angles as illustrated in Plate III, and Fig. 111. Sometimes two angles placed, back to back are employed. In order to give greater vertical stiffness to these members a section like Fig. 86 is used, consisting of two pairs of angles laced together, the depth of the section being equal to that of the upper chord so that the connection with it may be made on both top and bottom. (See Plate VII , Chap. XI.) Laterals of this type are used in the Delaware river bridge. Occasionally in short spans the composition is modified by latticing two single angles instead of two pairs of angles. This form is used in the Monongahela river bridge at Rankin, Pa., the size of both angles being $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. The long span over the same river at Port Perry has adjustable rods.

The various sections described are used also for lower laterals of through bridges. Although adjustable laterals are still used occasionally in the upper system, only stiff diagonals are employed at present in the lower lateral system. This statement also applies to the bridge at Port Perry, whose lower laterals consist of two pairs of angles latticed together. As these laterals are not connected to the stringers, they are stiffened in a horizontal direction by means of four horizontal members of similar composition which are connected at their extremities to the laterals

at their quarter points, thus forming a rectangle in plan The laterals stiffen each other also by the connection at their centers. Attention is called to the forms of splices used for both upper and lower laterals on Plates III and VII.

The connections of the lower laterals to through trusses is often very eccentric, causing large horizontal bending moments in the ends of the floor beams. This is avoided, in the best designs, by using larger connecting plates, and by incurring the cost of somewhat greater inconvenience in field riveting. In the upper lateral system the effect of eccentricity is not so serious, since the stresses are smaller and the connection is made to the stiff upper chord. Let the student observe the character of the lateral connections in this respect on Plates III, IV, VI , and VII .

The construction of the upper lateral system in deck bridges is practically the same as that of the lower system in through bridges, and that of the lower system of deck bridges the same as that of the upper system of through bridges. Sometimes the lower laterals are omitted in alternate panels, while in other cases they are omitted entirely. The latter arrangement is adopted in the standard plans for pin-connected deck bridges on the Northern Pacific Railway.

The lateral struts which are perpendicular to the upper chords of through trusses form also a part of the transverse or sway bracing. Sometimes the rest of the sway bracing consists merely of brackets connecting the lateral struts to the posts of the trusses, while at other times this is connected to a lower or intermediate strut by means of two or more web members as shown in the next article. In short spans the lateral strut is composed of two pairs of angles placed back to back and laced together as in Fig. 86, its depth being equal to that of the upper chord to whose upper and lower flanges it

is riveted by connecting plates. (See Plates III and VII.) Occasionally the upper angles are placed with their horizontal flanges on the lower side, extended across the top of the chord and riveted directly to it. Where the upper chord is rather deep and the trusses are separated by double tracks, the angles are often placed in the corners of a rectangle as in Fig. 82, Art. 74, and laced on the four sides. Two channels laced together are occasionally used. Another form of section is that in which the lacing of the first form mentioned is replaced by a solid web, forming practically a small plate girder.

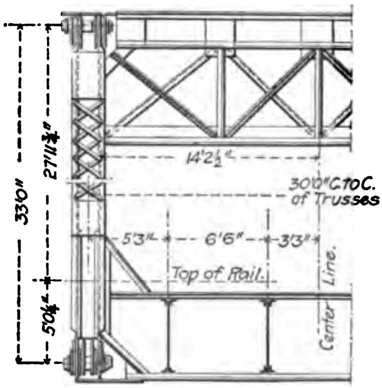
When the web connections of the sway bracing are rather close together, the lateral strut is sometimes reduced to a single pair of angles (Plate IV) or to one pair of angles with a web plate between, the latter form being shown on Plate VII . In double-track bridges this section is increased in stiffness horizontally by using bulb angles instead of the ordinary angles.

The composition of lower lateral struts in deck bridges comprises all the forms mentioned above except those containing a solid web plate with either one or two pairs of flange angles.

ART. 80. PORTAL AND SWAY BRACING.

When the required clearance extends to within two or three feet of the top of the lateral strut, the intermediate sway bracing of a through bridge consists merely in connecting the strut to the post at each end by means of a bracket or knee brace. (See Plates III and VI .) When there is more head room one of the simplest styles of bracing consists of a lattice girder, with a double system of webbing, as shown on Plate IV. The lower flange is placed as low as the head room will allow. With increasing depth four systems of webbing may be used, an example of which is given on Plate VII . For other examples, see the inset of the Engineering News, Jan. 11, 1900. Where

the depth is large, the lower strut is sometimes made like the upper or lateral strut. It will be noticed that the bracing on Plate VII also contains a small bracket. The use of brackets is generally confined to cases where the depth is small.



Section through L2. U2.

Fig. 103.

Another type is shown in outline in Fig. 103, and its details are given in Fig. 104. Sometimes the verticals are omitted in the webbing, thus reducing it to the Warren type of truss. The number of panels depends on the depth of the bracing and on the width of the bridge. An example of this form may be seen in Engineering Record, vol. 37, page 386, April 2, 1898.

The small connecting plates shown in the top view and section are intended to connect with a longitudinal strut which helps to stiffen the lateral struts in a horizontal direction, since it is also attached to the lateral diagonals at their intersection.

Fig. 105 shows two forms of intermediate sway bracing, one between the long posts of the trusses in which a quadruple system of diagonals is used, and the other between the sub-vertical struts with only two diagonals. In both cases the upper and lower struts are composed of a plate and a pair of bulb angles. In some cases the single pair of diagonals is used throughout the span, and occasionally a sub-vertical is suspended from the intersection of the diagonals to support the center of the lower strut. With further increase in depth the sway bracing is sometimes divided into two panels, one above the other, by means of an intermediate horizontal strut. In the Engineering

Record, vol. 44, page 467, Nov. 16, 1901, may be found an illustration of the sway bracing at the middle of the span of the Rankin bridge. The lateral strut consists of two pairs of angles $6'' \times 4'' \times \frac{3}{8}''$, connected by a system of double intersection lacing with $3'' \times 2'' \times \frac{5}{16}''$ angles; the other two struts consist

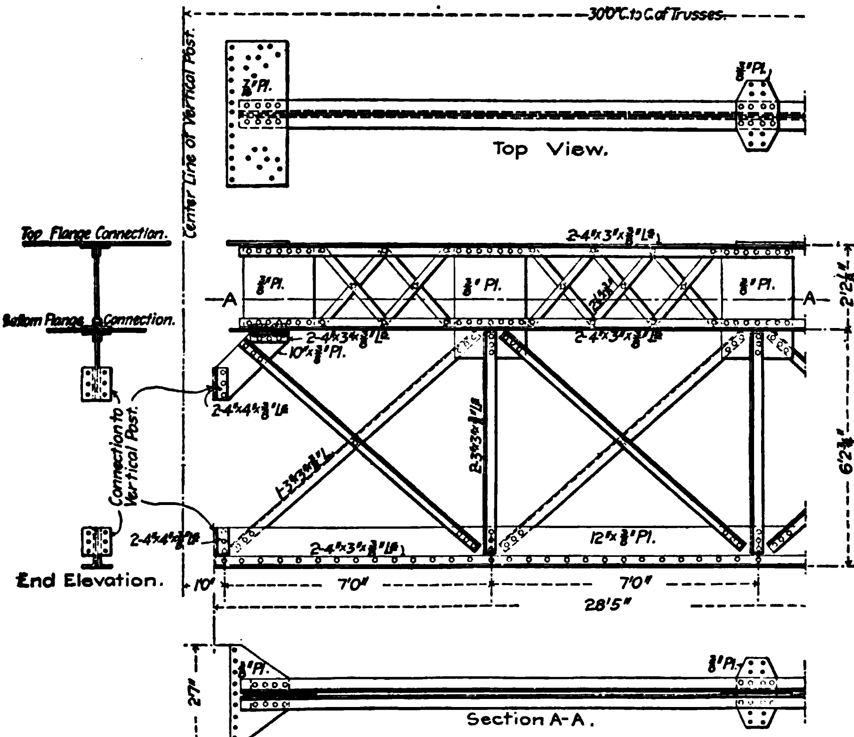


Fig. 104.

of two pairs of $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles laced with bars so as to be 18 inches deep, and the two diagonals in each panel are composed of two $3'' \times 3'' \times \frac{3}{8}''$ angles placed back to back. Toward the end of the span the bracing has only one panel, and at some intermediate points a single pair of diagonals crosses the two panels intersecting the middle strut at its center.

In the shorter spans of the Victoria Jubilee bridge at Montreal the upper strut is made up of two pairs of $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles laced, the lower strut of one web plate, $10'' \times \frac{3}{8}''$, and two flange angles, $6'' \times 3\frac{1}{2}''$, each of the two intersecting diagonals of one angle, $4'' \times 3'' \times \frac{3}{8}''$, and the sub-vertical of one angle, $3'' \times 3'' \times \frac{3}{8}''$. In the long span the composition is the same except that the angles and plate are increased in size.

The general character of the sway bracing of deck bridges is about the same as for through bridges. One example of both the intermediate and the end sway bracing is shown on Plate V. Another example may be found on the inset of the Engineering News, Nov. 29, 1900, and a third one in the Engineering Record, vol. 41, pages 125 and 126, Feb. 10, 1900.

Adjustable rods are still used to some extent in the sway bracing of both through and deck bridges, but the practice is not generally regarded with favor.

A number of the forms employed for intermediate sway bracing are also used in portal bracing, the details being made stronger, however, on account of the greater duty of the latter. Plate IV shows a portal having flanges with unequal-legged angles of ample size, and with deep plates to receive the connections of the web members, which consist of two systems of diagonals. The wide plates are continued around the ends of the portal, and extended into the bracket, so as to make a very rigid connection with the inner sides of the end posts. As indicated on the plate, this is a standard design of the Northern Pacific Railway. In the reference to Engineering Record mentioned in the preceding paragraph may be seen the view and details of a portal only about $4\frac{1}{2}$ feet deep at the middle, with double intersection webbing. The lower flange is curved down at the ends to form the flanges of the brackets, and solid web plates form the bracing in the end panels. The standard portal of the

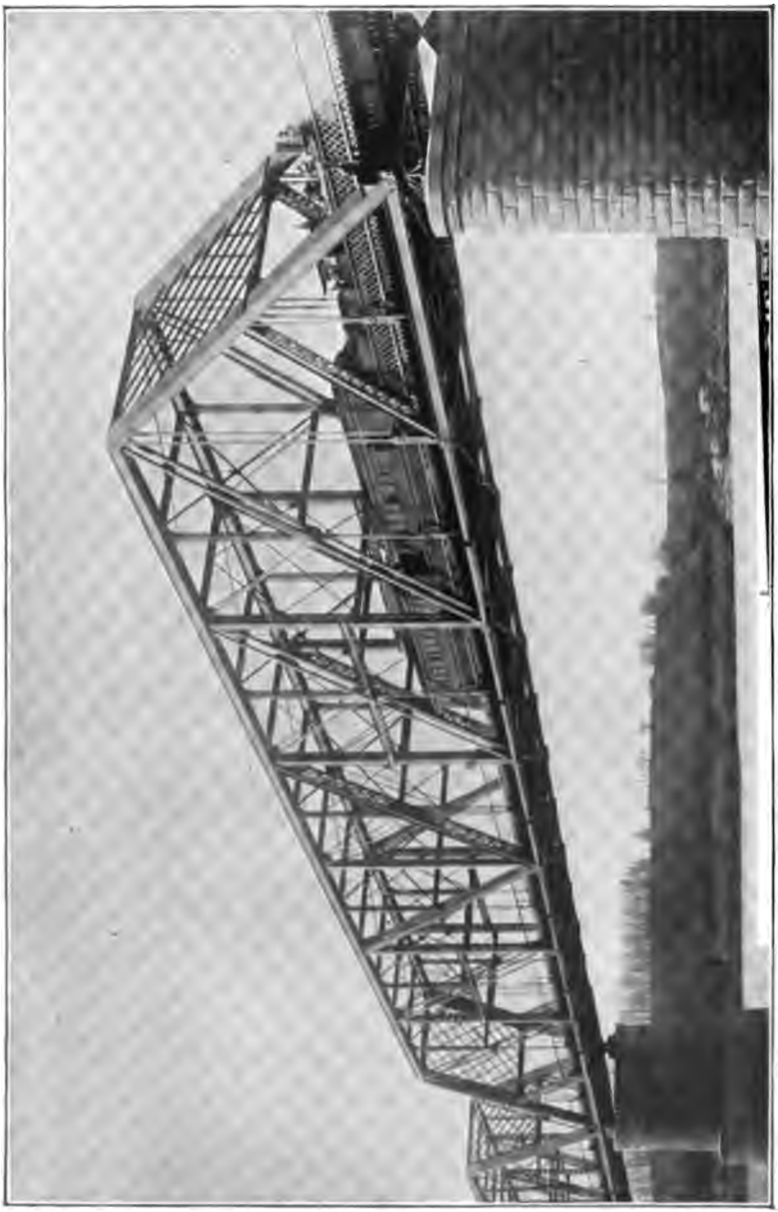


Fig. 105. Double-track Through Bridge over the Missouri River at Bellefontaine, Mo.

New York Central and Hudson River Railroad is given on Plate VII , Chap. XI.

In some cases where the head room is limited the portal bracing consists practically of two complete double intersection lattice girders with their connecting end brackets, one riveted to the top and the other to the bottom flanges of the end posts, the corresponding flanges of both girders being united by lacing. (See Fig. 6, Art. 3.) A plate is sometimes substituted for the upper lacing. An example of a double portal bracing, but of somewhat different design, is shown on Plate VI . Under similar conditions of limited head room the portal bracing is occasionally composed simply of a plate girder and of brackets with solid webs.

Perhaps the best illustration of the application of a lattice portal bracing to a bridge of long span is that of the Bellefontaine bridge shown in Fig. 105. The top strut consists of a web plate $30'' \times \frac{1}{2}''$ and two bulb angles $9'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, the lower strut of one plate $27'' \times \frac{1}{2}''$, one angle $4'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, and one bulb angle $9'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, and each of the twenty diagonals of a single $5'' \times 5'' \times \frac{1}{2}''$ angle. The plates extend around both sides of the bracing similar to that on Plate VII, and with neatly rounded corners.

The present practice in the design of portals for bridges whose depth affords adequate room consists in using relatively few members with sufficient strength to secure that degree of lateral rigidity which is now regarded as so essential. The members are all made of the same depth as the end posts, so as to permit them to be riveted to both the top and bottom flanges of the end posts. An excellent example of such a design is the portal of the United States bridge at Rock Island, Ill. The view given in Fig. 106 is that of the portal of the draw span, but it has the same construction as those used on the fixed spans. In the $216\frac{1}{2}$-foot fixed spans the lower strut has one

Fig. 106. Portal of United States Bridge at Rock Island, Ill.

Fig. 107. Portal of Delaware River Bridge, Philadelphia.

cover plate $16\frac{1}{2}'' \times \frac{3}{8}''$, four angles $3'' \times 3'' \times \frac{3}{8}''$, and is laced on three sides. The diagonals have two pairs of angles $5'' \times 3'' \times \frac{3}{8}''$, with one line of lacing. The upper strut has an upper cover plate $17\frac{1}{4}'' \times \frac{3}{8}''$, three angles $3'' \times 3'' \times \frac{3}{8}''$, one angle $4'' \times 4'' \times \frac{3}{8}''$, and a lower cover plate $7'' \times \frac{3}{8}''$. It is laced on two sides, one side being perpendicular to the flanges of the end post, and the other in the plane of the beveled end of the end post. The provision of connecting plates with curved edges indicate that some attention was paid to æsthetic considerations in this design.

The portal of the Delaware river bridge near Philadelphia is divided into two panels, one above the other. Fig. 107 indicates that the lower strut is practically a plate girder whose depth equals that of the end posts, while the middle strut and the diagonals consist of two pairs of angles laced together. The top strut is of novel design. In composition it resembles that of an upper chord member, but the two web plates are respectively perpendicular to the flanges of the end post and of the upper chord, and both the cover plate and the lower lacing are bent to the angle made by the end post with the adjoining upper chord member. Square connections could thus be made on one side with the portal diagonals and on the other side with the top laterals, which also consist of two pairs of angles laced as deep as the chords.

Another portal containing some new details is that of the Union Railroad Bridge at Rankin, Pa., shown in Fig. 108. Both the upper and the lower struts consist practically of two plate girders whose flanges, each having only one angle, are extended across the end posts, and riveted to them on the upper and lower sides respectively. The girders have their corresponding flanges laced together with a single system of diagonals composed of single angles. Double triangular brackets with solid webs and connecting plates are also used. In addition to the diagonals of the portal, a strut of the same composition as

the diagonals connects the middle of each horizontal strut with the intersection of the diagonals.

The portal bracing of the bridge erected by the same railroad at Port Perry differs from this one by substituting for the strut

Fig. 108. Portal of Rankin Bridge.

just mentioned one of only two angles laced in a similar way, but extending horizontally across the intersection of the diagonals, and riveted at each end to the top and bottom of the end post. The upper strut is also different in containing only four angles laced on the four sides.

This bridge contains the unusual feature of a double plate-girder portal bracing, connecting the feet of the end posts on their top and bottom flanges. The web plates of each of these girders are not continuous, but are connected by angles to the webs of the stringers, and thus to each other. The flanges, however, are continuous and are field-riveted to the webs. They consist of single angles. The function of this bracing is performed in many other bridges by an end floor beam riveted to the end posts.

The composition of the sway and portal bracing of the Victoria Jubilee bridge is given in Engineering Record, vol. 38, page 488, Nov. 5, 1898. Each of these contains only two struts and two intersecting diagonals.

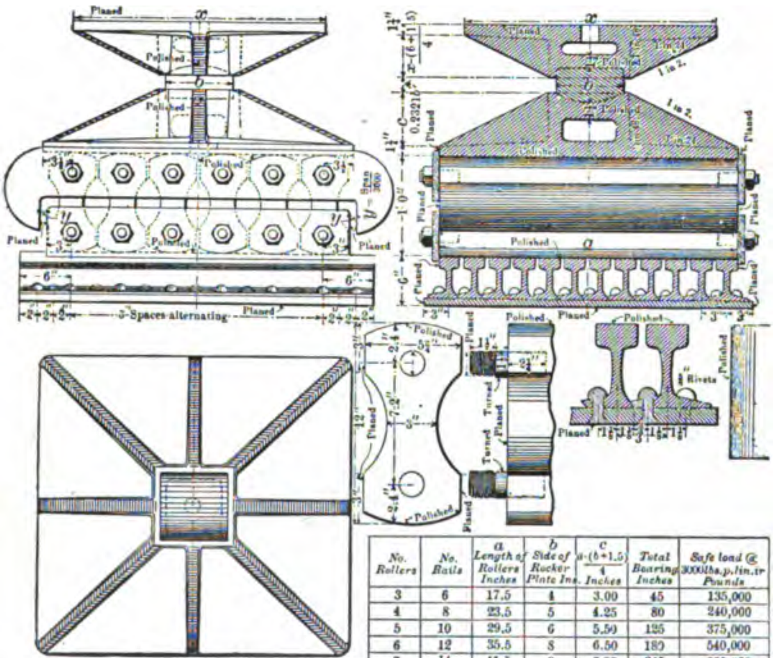
ART. 81. EXPANSION BEARINGS.

Pedestals, friction rollers, and bed plates, similar to those described in Art. 44, are used also for truss bridges. Two examples of expansion bearings containing cylindrical rollers are given on Plate III and in Fig. 111.

Complete detail drawings of pedestals, nests of cylindrical rollers, and rail plates, together with the castings for the fixed bearings, may be found on the insets of the Engineering News for Jan. 5 and Feb. 2, 1899. The nests contain 7 rollers each, their diameters being $4\frac{1}{2}$ and $4\frac{7}{8}$ inches respectively.

Fig. 109 shows the details of a standard expansion bearing, designed by GEORGE S. MORISON, which contains some valuable improvements over those employed in Europe. The steel rollers are 12 inches in diameter and spaced 6 inches between centers. The sides are parallel near the top and bottom, and hollowed out along the middle to facilitate cleaning with a brush. Contact between the parallel sides of the rollers prevents them from tipping over, but an additional provision against it is afforded

by means of the side plates, which engage stud bolts screwed into the ends of the rollers. The clearance between the hook of the upper plate and the square ends of the lower plate allows a linear movement of $y =$ span $/3000$ in both directions from the mean position. The rollers rest on a rail plate consisting of



No. Rollers	No. Rails	a Length of Rollers Inches	b Side of Rocker Plate Ins.	c $\frac{a-(b+1.5)}{4}$ Inches	Total Bearing Inches	Safe load @ 3000 lbs. p. lin. in. Pounds
3	6	17.5	4	3.00	45	135,000
4	8	23.5	5	4.25	80	240,000
5	10	29.5	6	5.50	125	375,000
6	12	35.5	8	6.50	180	540,000
7	14	41.5	9	7.75	245	735,000
8	16	47.5	10	9.00	320	960,000
9	18	53.5	11	10.25	405	1215,000
10	20	59.5	12	11.50	500	1500,000
11	22	65.5	13	12.75	605	1815,000
12	24	71.5	14	14.00	720	2160,000

Fig. 109.

T-rails riveted to a plate, with their tops afterward planed and polished. In large bridges the rail plate is bolted to the cast base, which is directly supported by the pier masonry. As the dust accumulates it is readily removed by passing a long-handled brush between the rails.

On top of the rollers rests a steel casting with its lower surface polished, and this in turn supports another casting by means of a block of polished steel called a rocker plate. The rocker plate fits into a socket in each casting, the surfaces of contact being segments of horizontal circular cylinders, whose axes are respectively parallel and perpendicular to the direction of the rollers. The radius of curvature in each case equals the length of one side of the square rocker plate. The upper casting sustains the pedestal, which in turn supports the end pin of the truss, and the connecting bolts pass also through the flanges of the stiff lower chord. The object of the rocker plate is to allow the bridge to adjust itself when erected, so that the bearing on the rollers, and hence also that on the bed plate, may be uniform. This eliminates the unequal distribution of load, which would otherwise be caused by imperfect workmanship in the construction of the truss and its supports. To secure the transverse stiffness of the lower ends of the end posts, they are preferably connected by an end floor beam which is riveted to them after the bridge is swung. The side plates project above and below the rollers respectively, thus acting as guides to prevent any lateral movement of rollers or casting.

In the vertical line of dimensions in Fig. 109 the next to the highest one should read $\frac{x-(b+1.5)}{4}$, while the value 0.2321 b does not belong to c, but to the dimension directly above c. The safe load given in the table is that recommended by the designer, no addition being made to the live load for impact. The allowance for impact is included in the unit stress adopted. See Transactions of American Society of Mechanical Engineers, vol. 15, page 153, 1894, and vol. 16, page 724 1895, for an account of the evolution of this bearing and some illustrations of its application. The description and detail drawings of a modified form of this bearing, in which a pin casting takes the place of

the usual bolster and of the top casting and rocker plate, thus materially reducing the height required, may also be found in Engineering Record, vol. 32, page 93, July 6, 1895.

In order to avoid the danger of the rollers getting out of place under the frequent jars to which the lighter bridges are subject, another improvement has been added by fitting steel plates into grooves cut in the ends of the middle roller, the plates projecting beyond the surface of the roller and forming teeth to engage spaces cut into the rail plate below and the bearing plate above. The details of this device may be seen on Plate II.

A side elevation of the fixed and expansion bearings of the Davenport, Rock Island and Northwestern Railroad bridge at Rock Island, Ill., may be seen in the inset of Engineering News, Jan. 11, 1900. This is a different type of bearing from the standard described above. The I-beams extend under both bearings over the full width of the pier.

The 6-inch segmental rollers of the International bridge at Buffalo are illustrated in Engineering Record, vol. 43, page 567, June 15, 1901. They are 3 inches wide, but have cylindrical spaces 6 inches long cored out of their sides and separated by $\frac{3}{4}$-inch vertical webs. The rollers are 3 feet 5 inches long.

Fig. 110 gives the details of the 12-inch rockers with parallel sides used in the truss shown on Plate V. The center roller has a spur or gear tooth at each end on both top and bottom, and these enter slots in the roller bed plate and the shoe respectively, thus retaining the rollers in their proper position. Grooves in the centers of the rollers engage longitudinal center strips to prevent the trusses from shifting sideways. The figure also indicates the construction of the pedestal and bed plate at the fixed end of the span.

In the Delaware river bridge seven segmental cast-steel rollers 18 inches in diameter are used to take care of a truss

reaction of 1200 tons. The rollers are of the same type as those mentioned in the preceding paragraph, and are $8\frac{1}{2}$ inches wide and 8 feet $2\frac{1}{2}$ inches long. The gear teeth on the middle roller are 7 inches long. A view of the fixed and expansion bearings on one pier is shown in Engineering Record, vol. 40, page 596, Nov. 25, 1899.

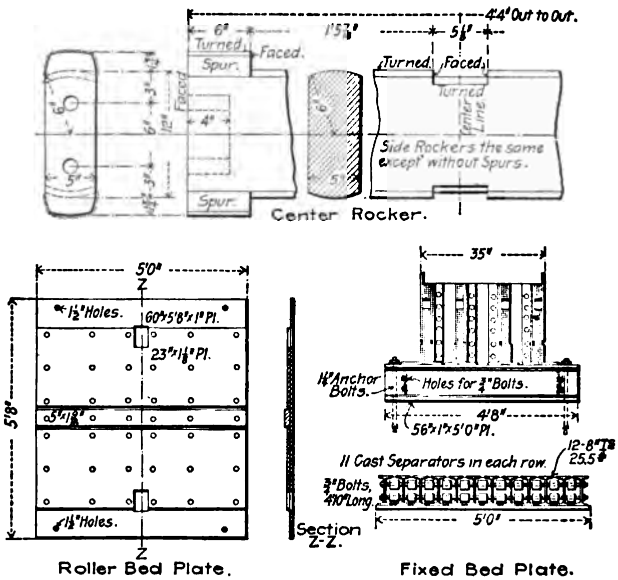


Fig. 110.

ART. 82. RAILROAD PIN BRIDGES — REFERENCES.

Variations in the composition of members throughout the span, the relations between the forms and dimensions of connecting members, and many of the smaller details related to the connections at the joints can best be studied by consulting the general drawings of different trusses. The following references are given to enable the student to become familiar with recent practice in these respects, no reference being included whose date precedes 1895, and only a few that are earlier than 1898.

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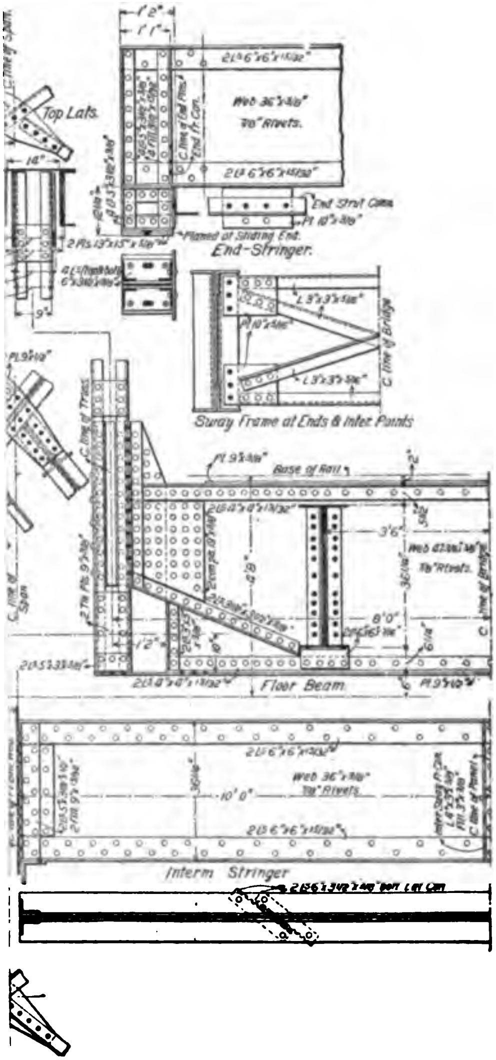

OF MT. STERLING, OHIO.

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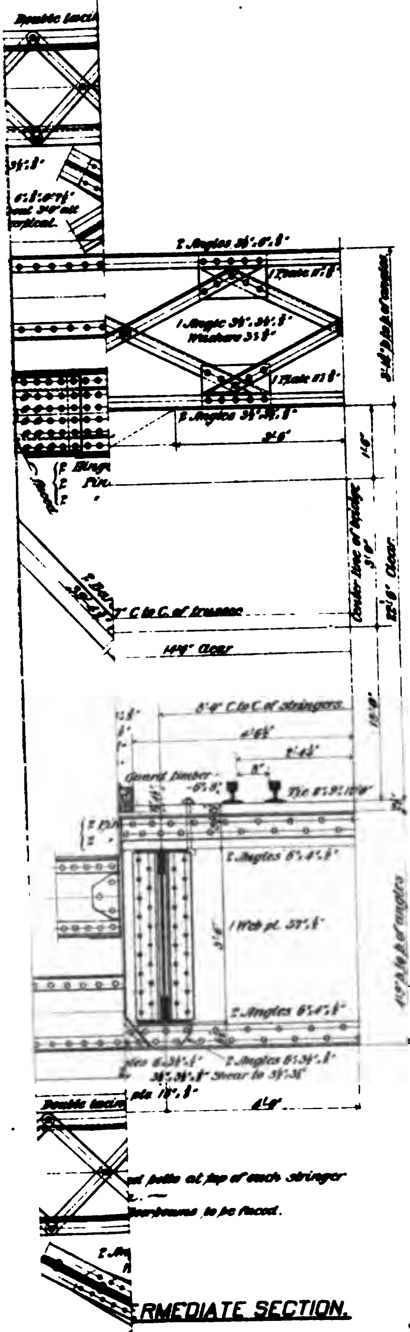
R-10-1078.



RMEDIATE SECTION.

BILL OF TRACK MATERIAL
FOR ONE SPAN.

Material	No. Reqd.	Size	Length	Mark
Cross ties	134	9"x 8"	12'-0"	
Guard timber	22	6"x 8"	16'-0"	
Std. Floor bolts	20	¾" dia.	1'-8½"	B ☐
Hook bolts	88	¾" dia.	1'-1½"	C ☐
Boat Spikes	248	⅝" dia.	0'-10"	
☐ See standard drawing R-10-1078				

Total weight of one span including track bolts and bearings. – 347200 lbs.

¾" Standard Floor bolt. One to every splice on guard timber.
¾" " Hook " One to every third tie – to each Stringer
⅝"x10" Boat Spikes – One to every tie, except at guard timber splices.

Note
Dimensions given for packing are exact. –

N.P.R'Y.
STANDARD PLAN.
160 FT. THROUGH SPAN.

SCALE ¾"–1'

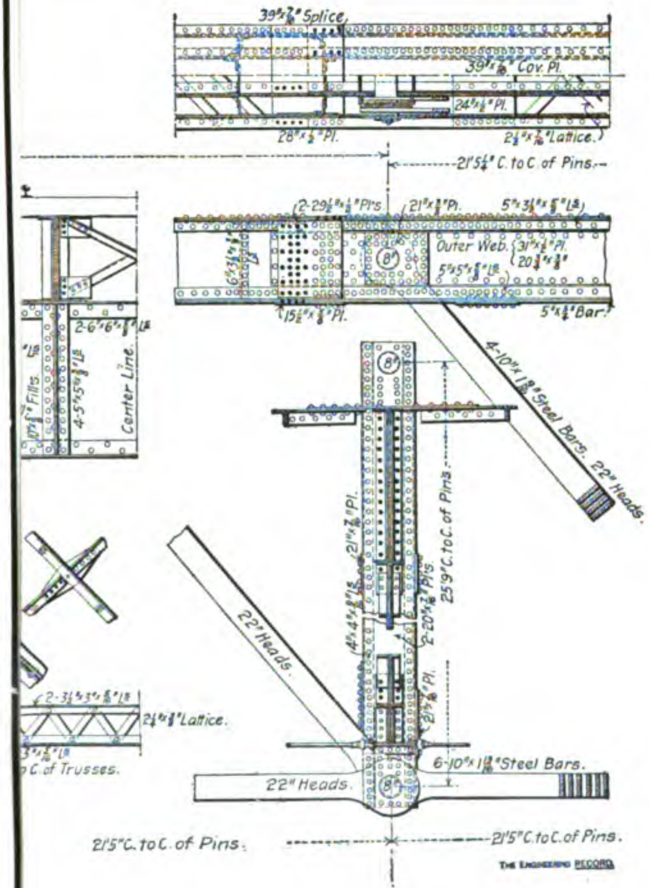
Approved.

Chief Engineer

RALPH MODJESKI,
CIVIL ENGINEER
Monadnock Bldg.
CHICAGO.
May – 1904.

LITHO. BY CHAS. HART 36 VESEY ST. N.Y.

ilroad.



RACK DECK BRIDGE OVER THE SCHUYLKILL

RD AVENUE, PHILADELPHIA.

INCHES. ERECTED IN 1897.

ls in Arts. 71, 73, 74, 77, 78, 80, and 81.

Recent Small Bridges on the Baltimore & Ohio Railroad. R. R. Gaz., v. 27, p. 34, Jan. 18, 1895.

Trunk Line Deck Bridge. Eng. Rec., v. 33, p. 58, Dec. 28, 1895.

Victoria Jubilee Bridge at Montreal; Grand Trunk Railway. Eng. News, v. 38, p. 130, Aug. 26, 1897; Eng. Rec., v. 38, p. 488, Nov. 5, 1898.

Superstructure of the Delaware River Bridge at Bridesburg, Philadelphia, for the Pennsylvania & New Jersey Railroad Company. By PAUL L. WÖLFEL. Proc. Engrs. Club of Phila., 1897, v. 14, p. 154; Eng. Rec., v. 40, p. 594, Nov. 25, 1899.

New United States Rock Island Bridge. Eng. Rec., v. 37, p. 384, Apr. 2, 1898; Eng. News, v. 36, p. 406, Dec. 17, 1896.

Newport and Cincinnati Bridge. Eng. Rec., v. 37, p. 448, Apr. 23, 1898.

Standard Plans for 120-foot Pony-truss Bridges. Northern Pacific Railway. Eng. News, v. 41, p. 14, Jan. 5, 1899. Standard Plans for 130-foot Through Truss Bridges. Eng. News, v. 41, p. 69, Feb. 2, 1899.

Complete detail drawings. These standards were superseded by later standards referred to in Art. 102, but they will furnish the student a good opportunity for comparative study. The 120-foot truss is replaced in the new standards by a through riveted truss.

Bridge 69, New York Division, Pennsylvania Railroad. Eng. Rec., v. 39, p. 371, Mar. 25, 1899.

Description and partial detail drawings of one of the spans of the double-track deck bridge over the Schuylkill River near Girard Avenue, Philadelphia. The illustrations are reproduced in Plate V, and in Fig. 110. Many of the details are referred to in the preceding articles of this chapter. The structure was designed for heavy traffic under comparatively high speeds. (See Proc. Engrs. Club of Phila., v. 14, p. 302, Jan., 1898, for an account by JOSEPH T. RICHARDS of the operation of moving aside the old Whipple truss bridge and putting this new bridge in its place in two minutes and twenty-eight seconds, on Oct. 17, 1897.)

Short-span Railroad Bridges. Eng. Rec., v. 40, p. 717, Dec. 30, 1899.

Davenport and Rock Island Bridge over the Mississippi River. Eng. News, v. 43, p. 26, Jan. 11, 1900.

Lehigh Valley Railroad Bridge at Easton, Pa. Eng. Rec., v. 41, p. 124, Feb. 10, 1900.

Bridge Work on the Baltimore & Ohio Railroad. Eng. Rec., v. 41, p. 271, Mar. 24, 1900.

Special Bridge and Viaduct Construction in Western Pennsylvania. Eng. Rec., v. 41, pp. 465 and 516, May 19 and June 12, 1900.

Terminal Improvements of the Chesapeake & Ohio Railway at Richmond, Va. Eng. News, v. 44, p. 379, Nov. 29, 1900.

Northern Pacific Standard Bridge Plans. By RALPH MODJESKI. Jour. W. Soc. Engrs., v. 6, p. 51, Feb., 1901.

Reconstruction of the Glasgow Bridge on the Chicago & Alton Railway. By W. D. TAYLOR. Eng. Rec., v. 43, p. 241, Mar. 16, 1901.

International Bridge, Buffalo. Eng. Rec., v. 43, p. 566, June 15, 1901.

The Rankin Bridge. Eng. Rec., v. 44, p. 465, Nov. 16, 1901.

Die Brücke der Pennsylvania-Eisenbahn über den Delaware bei Philadelphia. Von F. C. KUNZ. Allgemeine Bauzeitung, Wien, Heft. 1, 1901.

Description of the design, construction, and erection, illustrated by numerous views and a number of large plates showing many of the details of the fixed and swing spans. Analyses of their weights are included. (See Figs. 10 and 107.)

Chesapeake & Ohio Railroad Bridge at Richmond, Va. Eng. Rec., v. 45, p. 290, Mar. 29, 1902.

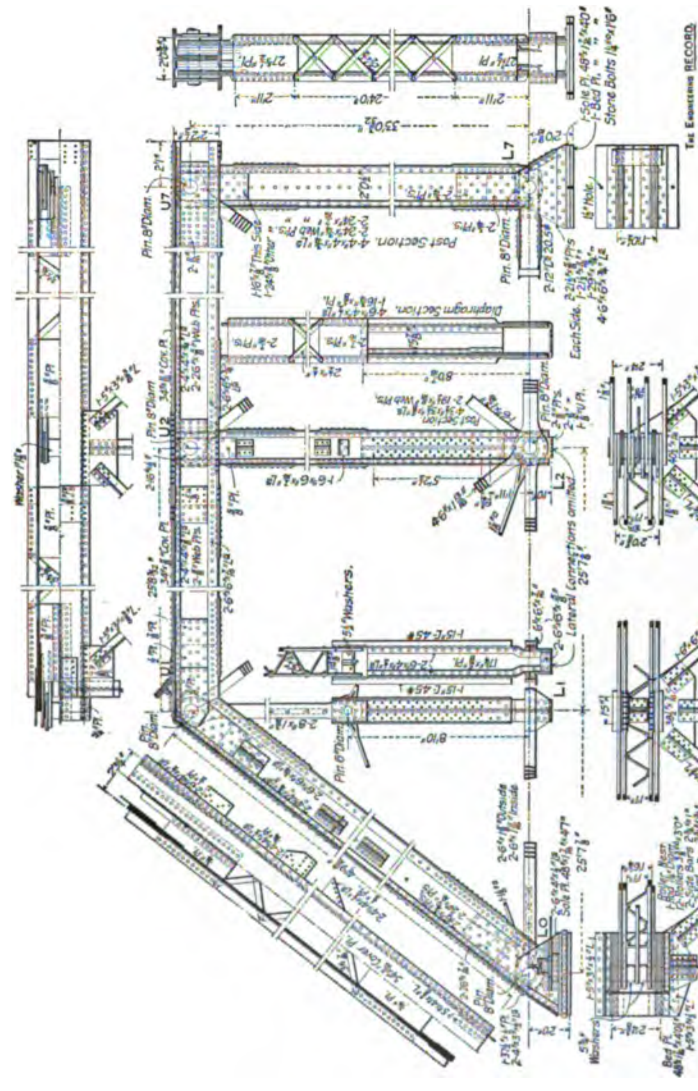


Fig. 111. Union Railroad Double-track Through Bridge over the Pittsburg, Virginia and Charleston Railroad at Port Perry, Pa. Span, 177¾ feet. Built in 1897. See references to various details in Arts. 73, 74, 77, 79, 81, and 96

Heavy Double-track Skew Bridge. Eng. Rec., v. 45, p. 435, May 10, 1902.

Four-truss Double-deck Railroad Bridge. Eng. Rec., v. 46, pp. 338 and 364, Oct. 11 and 18, 1902; v. 49, p. 170, Feb. 6, 1904.

Pennsylvania Railroad Bridge at Fifty-second Street, Philadelphia. Eng. Rec., v. 46, p. 398, Oct. 25, 1902.

Standard Plans for Bridges on the Atchison, Topeka & Santa Fé Railway. Eng. News, v. 49, p. 482, May 28, 1903.

Plattsmouth Bridge of the Burlington. R. R. Gaz., v. 35, p. 564, Aug. 7, 1903; Eng. Rec., v. 50, p. 240, Aug. 27, 1904.

Five-hundred-foot Channel Span of the Clairton Bridge. Eng. Rec., v. 49, pp. 323 and 383, Mar. 12 and 26, 1904.

New Westminster Bridge over the Fraser River, British Columbia. Eng. Rec., v. 49, pp. 544, 582, and 644, Apr. 30, May 7 and 21, 1904; Eng. News, v. 53, pp. 611 and 647, June 15 and 22, 1905.

Woodsville Railroad and Highway Bridge. Eng. Rec., v. 49, p. 793, June 25, 1904.

Kaw River Bridge of the Chicago Great Western and the Missouri Pacific. R. R. Gaz., v. 37, p. 195, July 29, 1904; Eng. Rec., v. 55, p. 395, Mar. 23, 1907.

Standard Bridges on the Harriman Lines. R. R. Gaz., v. 39, pp. 224, 256, 274, and 319, Sept. 8, 15, 22, and Oct. 6, 1905.

New Bismarck Bridge of the Northern Pacific. R. R. Gaz., v. 40, pp. 174 and 197, Feb. 23 and Mar. 2, 1906; Eng. Rec., v. 53, p. 231, Feb. 24, 1906.

All-steel Open-floor Railroad Bridge. Eng. Rec., v. 53, p. 777, June 23, 1906.

Fig. 111¼. Baltimore and Ohio Railroad Bridge over Patapsco River, at Ilchester, Md. Built in 1903.

Reconstruction of the Parkersburg Bridge on the Baltimore & Ohio Railroad. Eng. Rec., v. 55, p. 318, Mar. 9, 1907.

Big Lazer Creek Bridge. Eng. Rec., v. 57, p. 716, June 6, 1908.

Replacing the Clyde River Bridge of the West Shore Railroad. Eng. Rec., v. 58, p. 463, Oct. 24, 1908.

River Spans of the St. Louis Municipal Bridge. Eng. Rec., v. 60, p. 487, Oct. 30, 1909; v. 62, p. 641, Dec. 3, 1910.

Superstructure of the McKinley Bridge at St. Louis. By H. M. MORSE. Eng. Rec., v. 61, p. 572, Apr. 30, 1910; Eng. News, v. 64, p. 85, July 28, 1910.

Superstructure of the Mobridge Bridge on the Chicago, Milwaukee & St. Paul Railway. Eng. Rec., v. 61, p. 762, June 11, 1910.

Design and Erection of the Missouri River Bridge for the Pacific Extension of the C. M. & St. P. Ry. By J. H. PRIOR. Eng. News, v. 64, p. 196, Aug. 25, 1910.

Miles Glacier Bridge, Alaska, on the Copper River & Northwestern Railway. Eng. Rec., v. 62, p. 153, Aug. 6, 1910.

CHAPTER IX.

DESIGN OF A PIN TRUSS BRIDGE.

ART. 83. SPECIFICATIONS.

Let it be required to design a single-track through railroad bridge whose trusses are of the Pratt type and whose span is 175 feet between centers of end pins. The cross-ties, foot planks, and guard timbers are to be of long-leaf Southern yellow pine, the truss pins of medium steel, and the rest of the structure of soft rolled steel.

The trusses are to be spaced 17 feet center to center and the clear opening is not to be less than that shown in Fig. 112.

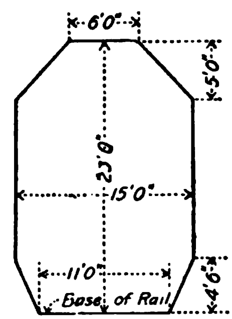


Fig. 112.

LOADS.

The live load is to be class Q of WADDELL'S compromise standard system (Art. 32), but the equivalent live loads given on the diagrams in WADDELL'S specifications are to be used instead of the actual wheel concentrations.

The effect of impact and vibration shall be added to the maximum stresses resulting from the above live load, and is to be determined by the following formula:

$$I = S\left(\frac{300}{L + 300}\right),$$

in which I is the impact, S the computed maximum live-load stress, and L the length of the loaded distance in feet which produces the maximum stress in the member.

To provide for wind stresses due to the pressure of the wind on the truss and train, as well as for lateral vibrations from high-speed trains, the wind load

on the lower lateral system of through bridges shall be 600 pounds per linear foot, 450 pounds of this to be treated as a moving load, and as acting on a train of cars at a line 6 feet above base of rail; and the static wind load on the upper lateral system shall be 150 pounds per linear foot.

The total traction load on any portion of the bridge is to be taken as 20 percent of the greatest live load that can be placed on that portion. No percentage of impact is to be added to traction loads.

UNIT STRESSES.

No metal less than three-eighths of an inch in thickness shall be used except for filling plates.

All parts of the structure shall be so proportioned that the sum of the maximum loads, together with the impact, shall not cause the tensile stress to exceed 15 000 pounds per square inch. The same limiting unit stresses shall also be used for members stressed by wind pressure, or momentum of moving train. Net sections must be used in all cases in calculating tension members, and, in deducting rivet holes, they must be taken one-eighth of an inch larger than the size of the rivets. The net section of any tension flange or member shall be determined by a plane cutting the flange square across at any point. The greatest number of rivet holes which can be cut by any such plane, or whose centers come nearer than two and a half inches to said plane, are to be deducted from the gross section when computing the net area.

For compression members, this permissible stress of 15 000 pounds shall be reduced in proportion to the ratio of the length to the least radius of gyration of the section by the following formula:

$$p = \frac{15\,000}{1 + \frac{1}{13\,500} \cdot \frac{l^2}{r^2}},$$

in which p is the permissible working stress per square inch in compression, l the length of piece in inches, between centers of connections, and r the least radius of gyration of the section in inches. No compression member, however, shall have a length exceeding 100 times its least radius of gyration, except those members of trusses whose main function is to resist tension and all compression members for wind bracing which may have a length not exceeding 120 times the least radius of gyration.

Members subject to alternate stresses of tension and compression in immediate succession (as counter stresses in web members of trusses) shall be so proportioned that the total sectional area is equal to the sum of the areas required for each stress.

In case the maximum stresses due to wind added to the maximum stresses due to vertical loading (including impact) shall exceed 19 000 pounds per square inch, properly reduced for compression, addition must be made to such sections until this limit is not exceeded. The permissible stresses for the connections shall be increased proportionately. Should the stresses be reversed in any possible case, proper provision must be made for such stresses in the opposite direction.

To insure the stability of bridges under increased live loads, a live load shall be assumed 100 per cent greater than that previously provided for in this specification. If the resultant stress per square inch in any member is more than twice the permissible stress previously specified, additions must be made to the sections until that limit is not exceeded.

The shearing stress on rivets, bolts, or pins shall not exceed 11 000 pounds per square inch of section; and the pressure upon the bearing surface of the projected semi-intrados (diameter times thickness) of the rivet, bolt, or pin hole shall not exceed 22 000 pounds per square inch. In field riveting, the number of rivets thus found shall be increased 25 percent if driven by hand, and 10 percent if satisfactory power riveters are used. The amount of field riveting must be reduced to a minimum, without, however, diminishing the number of rivets requisite for strength and rigidity. Whenever it is practicable, all designs are to be so made that the field rivets can be driven readily. For members of any importance, more than two rivets are to be used for each connection. Rivets are not to be used in direct tension.

If the extreme fiber stress resulting from the bending due to the weight only of any member does not exceed 10 percent of the specified unit stress, the effect of such bending may be ignored; but if it does so exceed, its effect must be combined with those of the other stresses, using, however, for determining the sectional area, a unit stress 10 percent greater than that specified.

In general, all trusses shall have main end posts inclined. The effective length of pin-connected spans shall be the distance between centers of end pins of trusses. The effective depth shall be the perpendicular distance between gravity lines of chords, which lines must pass through the centers of pins.

GENERAL PRINCIPLES OF DESIGNING.

[From WADDELL'S Specifications.]

The axes of all members of trusses or girders, and those of lateral systems coming together at an apex of a truss or girder, must intersect at a point whenever such an arrangement is practicable; otherwise the greatest care must be employed to insure that all the induced stresses and bending moments caused by the eccentricity be properly provided for.

Truss members and portions of truss members must always be arranged in pairs symmetrically about the central plane of the truss, except in the case of single members the axes of which lie in said central plane of truss.

In proportioning main members of bridges, symmetry of section about two principal planes at right angles to each other is to be attained wherever practicable; but in designing top chords and inclined end posts, this rule cannot be followed.

In both tension and compression members the center line of applied stress must invariably coincide with the axial right line passing through the centers of gravity of all cross-sections of the member taken at right angles thereto.

The principle of symmetry in designing must be carried even into the riveting; and groups of rivets must be made to balance about center lines and central planes to as great an extent as is practicable.

In all structural metal work, excepting only the machinery for operating movable bridges, no torsion on any member shall be permitted if it can possibly be avoided; otherwise the greatest care must be taken to provide ample strength and rigidity for every portion of the structure affected by such torsion.

In all main members having an excess of section above that called for by the greatest combination of stresses, the entire detailing is to be proportioned to correspond with the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected. In this connection, though, the reduced capacity of single angles connected by one leg only must not be forgotten.

Designs must invariably be made so that all metal work after erection shall be accessible to the paint brush, excepting, of course, those surfaces which are in contact with each other or with the masonry. This requirement rules out all closed columns of every type and description.

In general, details must always be proportioned to resist every direct and indirect stress that may ever come upon them under any probable circumstances, without subjecting any portion of their material to a stress greater than the legitimate corresponding working stress.

In all designs simplicity in both main members and details is to be considered of the greatest importance. In all structures rigidity is to be considered quite as important an element as mere strength.

In all designing true economy must be given the utmost consideration; and no useless material must be employed, every pound of metal in the structure having a legitimate function; but economy of material must not be quoted as an excuse for using inferior details or scamping the work in respect to strength, rigidity, or appearance.

In all structural work the subject of æsthetics must be duly considered; and all designs are to be made in harmony with the principles thereof, to as great an extent as the money available for the work will permit, or as the environment of the structure calls for.

For convenience of reference the remaining items of the specifications are printed in the following articles to which they relate. In the extracts from specifications which are printed in this chapter, slight modifications are sometimes made for the sake of simplifying the problem for the student who is taking his first lessons in designing, rather than to express disagreement with the original provisions.

Seven panels will be adopted for the trusses, making the panel length 25 feet. Six panels would make the panel length over 29 feet, thus giving a comparatively heavy floor system; and since the spacing of the trusses is 17 feet, it would give poorer proportions to the lateral systems, and therefore render them less effective. On the other hand, eight panels make the panel length less than 22 feet, which is rather short for a bridge of this span.

ART. 84. FLOOR TIMBERS.

SPECIFICATIONS. — Cross-ties, foot planks, and guard timbers shall be of long-leaf Southern yellow pine. The wooden floor shall be so designed as to insure safety from passing trains for the railroad employees. The spaces between cross-ties shall, in general, not be less than five inches nor more than six inches wide. The sizes of the cross-ties shall be such as to give the requisite resistance to bending, under the assumption that the load on one pair of wheels is distributed equally over three ties, the effect of impact being considered. No cross-tie shall be less than seven, or preferably, eight inches wide, nor less than six inches deep, nor less than 10 feet long.

Cross-ties shall be notched not less than one-half inch over the stringers and be given a full and even bearing on the flanges; and each alternate cross-tie shall be secured thereto at each end by a ¾-inch hook bolt, having at the hook end a square shank, at least two inches long, to prevent the bolt from turning. All timber bolts shall be of soft steel with cold-pressed threads.

Outer guard timbers shall be 6″ × 8″, laid flat, notched one inch over the cross-ties, and placed so that their inner faces shall be just twelve inches from the gage planes of the rail. Each guard timber must be bolted to each alter-

nate cross-tie by a ¾-inch screw bolt, the head of which shall be countersunk into the wood by means of a cup-shaped washer. Each guard timber must be spliced over a cross-tie with a half-and-half joint of at least six inches lap, through which must pass a ¾-inch screw bolt. Guard timbers shall extend over all piers and abutments.

Inner guard rails shall consist of steel track rails securely fastened to the cross-ties, so that the outer sides of the heads shall be just five inches from the gage planes of the track rails.

The allowable tension in the extreme fibers of long-leaf Southern yellow pine timber in bending, the effect of impact being considered, shall be 2000 pounds per square inch.

In estimating the dead load the weight of yellow pine shall be assumed at 3¾ pounds per foot, board measure, and that of the rails, spikes, and joints at 160 pounds per linear foot of track.

The greatest stress in the cross-ties is produced by the alternative loading specified. (See Arts. 32 and 83.) The weight on one axle is 58 000 pounds. The impact is also 58 000 pounds. If the cross-ties be 8 inches wide and spaced 6 inches in the clear, three ties and spaces will cover a length of 3½ feet. Assuming the total weight of the track as 450 pounds per linear foot, the weight for a length of 3½ feet is 1575 pounds. The total load on three ties is therefore 117 575 pounds, and for each rail on one tie 19 600 pounds. The dead load is relatively so small that it may be assumed to be also concentrated at the track rails, without appreciable error. The stringers are spaced 6½ feet apart (Art. 85). The cross-tie is a beam with two concentrated loads, each of 19 600 pounds, spaced 4 feet 11½ inches apart and placed symmetrically with respect to the supports furnished by the stringers. The bending moment is therefore 181 300 pound-inches. For a unit stress of 2000 pounds per square inch and a width of 8 inches, the required depth of the cross-tie is found to be 8.24 inches. A depth of 9 inches will accordingly be taken. If the width were 7 inches, the depth would also be 9 inches, but this width makes the compression under the rail rather high.

Let the cross-ties be alternately 10 and 14 feet in length, the additional length being required to support a 2-inch plank footwalk on each side of the track. A computation of the weight of the track shows that it equals about 440 pounds per linear foot.

ART. 85. TRACK STRINGERS.

SPECIFICATIONS. — The stringers shall be spaced $6\frac{1}{2}$ feet between centers. Stringers for truss bridges shall invariably be built of plates and angles, and no cover plates will be allowed for the flanges. Their depths shall be made not less than the most economic ones in respect to weight of metal required, provided that the bridge clearance will permit, and never less than one-twelfth of the span. The stringers are to be riveted to the webs of the floor beams. No splices will be allowed in their flanges, nor any in their webs, provided that sufficiently long web plates are procurable. The compression flanges shall be made of the same gross section as the tension flanges; and they shall be so stiffened that the unsupported length shall never exceed twelve times the width of flange.

Rigid diagonal bracing of angles is invariably to be used between the top flanges of stringers, and rigid cross-frames are to be employed near all expansion points. If the panel length exceed thirty feet, there shall be a cross-frame at mid-length between the contiguous stringers of each track; but for all shorter panels the rigid lower lateral diagonals which are riveted to the bottom flanges will stiffen the latter sufficiently.

The effective length of the stringers shall be the distance between centers of floor-beam webs. The effective depth shall be the distance between the lines passing through the centers of gravity of the sections of the upper and lower flanges. The unit stress in the net section of the tension flange shall not exceed 15 000 pounds per square inch. The web shall be regarded as resisting its proportion of the bending moment. The shearing stress in the web plate shall not exceed 10 000 pounds per square inch. See other unit stresses in Art. 83. The web shall have stiffeners riveted on both sides, at intervals not exceeding the full depth of the web plate when its thickness is less than one-sixtieth of the unsupported distance between the flange angles. The end stiffeners are to be faced or otherwise treated so as to make the stringers of exact length throughout, and so as to effect a uniform bearing of the end stiffeners against the webs of the floor beams. The general rules for riveting applied to the design of plate girders (Chap. VII) are to apply also to stringers. Flanges of stringers carrying the vertical load from the cross-ties shall have their rivets spaced uniformly from end to end, and at the minimum distance employed.

Since the design of a plate girder is explained in detail in Chapter VII, and a stringer is a plate girder of short span and simple form, only the principal results of the computations are given in this article, with but very brief descriptions of the methods employed.

The span of the stringer equals the panel length of the truss, or 25 feet. The equivalent uniform live load for a span of 25 feet is 9850 pounds per linear foot per track, and the maximum bending moment for one stringer is 384 800 pound-feet. The coefficient of impact for the same span is 0.923, and the corresponding moment, 355 100 pound-feet. The weight of the track carried by one stringer is 220 x 25 = 5500 pounds, while the weight of one stringer and of half the lateral bracing is assumed to be 5100 pounds, making the dead load 10 600 pounds. The dead-load moment is 33 100, and the total bending moment 773 000 pound-feet.

The diagram of end shears gives 146 000 pounds for a span of 25 feet, making the vertical live-load shear at the end of each stringer 73 000 pounds. The allowance for impact is 67 380 pounds, and the shear due to dead load 5300 pounds, thus giving a total vertical shear of 145 680 pounds.

Since the specifications require the depth to be taken large enough to make the weight of metal a minimum, let the depth of the web plate be taken as 42 inches, or a little less than one-seventh of the span. The method of determining this value, as well as the approximate weight of the stringer, will be explained later in this article. Let the web project one-half inch above the upper flange angles, so that the cross-ties need to be notched only over the web plate and not over the full width of the flange.

For the specified shearing stress of 10 000 pounds per square inch the net section of the web must be 14.57 square inches.

A thickness of $\frac{7}{16}''$ allows for 9 rivet holes of $\frac{15}{16}''$ diameter, or only 8 of $1''$ diameter. This is probably just about sufficient for the net section, which is that through the inner line of rivets in the end connection of the stringer. This thickness, however, requires stiffeners to be used, which may be avoided by increasing the thickness to $\frac{1}{2}''$. As flange angles 6 inches wide are most suitable for stringers without cover plates, the clear distance between flange angles is about $29\frac{1}{2}$ inches. It will be economical to do this, since seven pairs of stiffener angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, weigh about 400 pounds, while the increased weight of the web plate is only about 220 pounds. The extra material in the web plate also increases the stiffness of the stringer.

The outer row of rivets in the 6-inch flange angles is $2\frac{1}{4}$ inches from the backs of the angles (Art. 34). According to the method explained in Art. 56, a section of the web plate passing through the outer rivets of the flanges has its resisting moment reduced to 88.4 percent of that of the solid plate, and hence one-sixth of this, or 14.7 percent of its gross section, may be used as equivalent flange area. This gives $0.147 \times 41.5 \times \frac{1}{2} = 3.06$ square inches, provided the half inch which the web plate projects above the top flange is neglected. If the section be taken through the inner rivets, which are $4\frac{3}{4}$ inches from the backs of the angles, the corresponding values are 91.4 percent, 15.2 percent, and 3.15 square inches. It will be observed that the ratios for the equivalent flange areas are a little over one-seventh.

For the specified tensile stress of 15 000 pounds per square inch and an estimated effective depth of 38.2 inches for the section through the outer rivet holes and the back of the lower flange angles placed $\frac{1}{8}''$ below the edge of the web, the required net area of the lower flange is

$$\frac{773\,000 \times 12}{38.2 \times 15\,000} = 16.18 \text{ square inches,}$$

and that of the flange angles alone is $16.18 - 3.06 = 13.12$ square inches. Two angles $6'' \times 6'' \times \frac{5}{8}''$ give a net area of $2(7.11 - 0.625) = 12.97$ square inches. If the section be taken through the inner rivet holes, the effective depth is 38.45 inches, and the required net area of the flange angles is

$$16.08 - 3.15 = 12.93 \text{ square inches.}$$

In order to avoid using angles $\frac{11}{16}''$ thick let the backs of the lower flange angles be placed $\frac{1}{4}''$ instead of $\frac{1}{8}''$ below the edge of the web plate. In this case the smaller effective depth becomes $41.5 + \frac{1}{4} - 1.73 - 1.68 = 38.34$ inches. This reduces the required net area from 13.12 to 12.97 square inches, and hence the $6'' \times 6'' \times \frac{5}{8}''$ angles may be accepted.

Let the rivet pitch in the flanges be determined next. The maximum vertical shear at the end is 145 680 pounds, and the increment of flange stress per linear inch (see Art. 59) is

$$\frac{12.97}{16.03} \times \frac{145\,680}{38.34} = 3075 \text{ pounds.}$$

The vertical load on the flange, including impact, is

$$52\,790/42 = 1257 \text{ pounds.}$$

The resultant of these horizontal and vertical components is 3325 pounds. The allowable bearing value of a $\frac{7}{8}''$ rivet in a $\frac{1}{2}''$ web plate is 9630 pounds, and hence the theoretic rivet pitch at the end is $9630/3325 = 2.90$ inches. In accordance with the specifications a uniform pitch of $2\frac{7}{8}$ inches will be used throughout.

Since the vertical angles which connect the end of the stringer to the web of the floor beam are to be straight, fillers whose thickness equals that of the flange angles are placed

under the connecting angles and made wide enough to receive an extra row of rivets beyond the angles. The value of a $\frac{7}{8}''$ rivet in single shear at 11 000 pounds per square inch is 6610 pounds, while the bearing value is 9630 pounds, as stated in the preceding paragraph. As the rivets connecting the angles and the fillers to the web plate are in double shear, the bearing value of the rivets will govern in determining the number required to transmit the shear from the web plate into the connections. This number is therefore $145\,680/9630 = 16$.

In finding the minimum number of rivets which must pass through the angles, the value of the rivets in double shear will govern, since that is less than their bearing in the two angles combined, or in the web and filler plates combined. The number required must therefore not be less than $145\,680/13\,220 = 11$. Those passing also through the flange angles cannot be counted in either number, since their duty is to transmit the flange stresses.

The rivets connecting the other legs of these angles to the web of the floor beam are field rivets, and since there are two rows of rivets in single shear, the number in each angle must be one-fourth greater than 11, which gives 14. While this number of rivets may be crowded into a single row, it makes the pitch about $2\frac{5}{8}$ inches, and it is therefore preferable to use $6'' \times 6''$ angles. By counting the rivets in the flange angles, the number in the other leg of each angle is 15, and as the rivets in the adjacent rows must stagger, it is necessary to put 15 rivets into the other leg also. The thickness of the connecting angles cannot be determined theoretically; but experience shows that $\frac{7}{16}''$ will be sufficient, although $\frac{1}{2}''$ is frequently employed. As the web is $\frac{1}{2}''$ thick and the stringer has to be faced at the ends, the angles will be made $\frac{1}{2}''$ thick. The filler plates must then be $9'' \times \frac{5}{8}''$. Fig. 113 shows that

all the necessary conditions are satisfied by placing 6 rivets in the fillers outside of the angles.

Sometimes fillers are used in practice whose width does not exceed that of the angles; but such an arrangement is not to be approved. If in the above example 16 rivets could be put through the connecting angles in the space between the

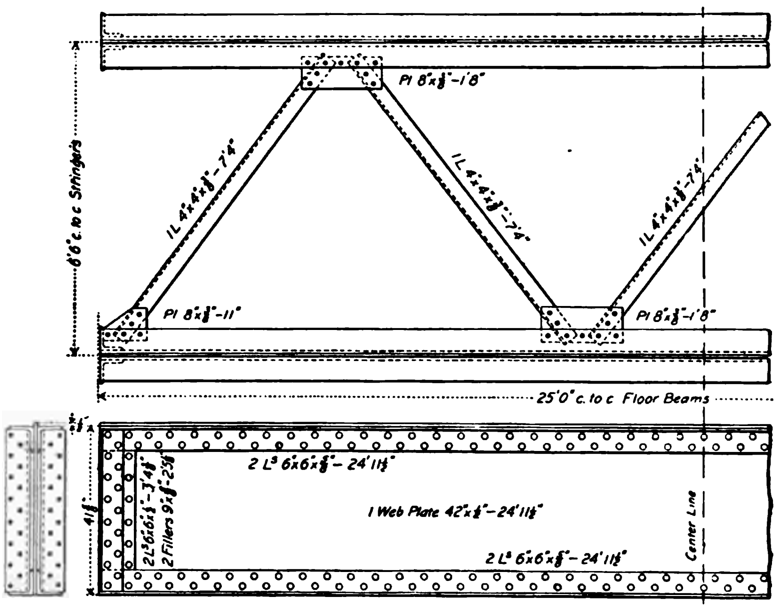


Fig. 113.

flange angles without reducing the pitch below the minimum allowed, the connection would not have the same degree of strength; for some tendency to bend the rivets is developed in transferring the entire shear directly from the web to the angles, to resist which extra rivets would be required. With the arrangement shown in Fig. 113 extra rivets are often necessary when only a single row of rivets is placed in the angles in order to keep the pitch from exceeding the maximum allowed in the row outside of the angles.

The skeleton diagram of the lateral system is given in Fig. 114. According to the specifications the upper flange must be stayed laterally at intervals not exceeding 12.56 feet, and this condition is satisfied by the arrangement shown. It also makes the connections with both girders exactly alike, so that the same patterns may be used for both. The stresses S_1 and S_2, due to

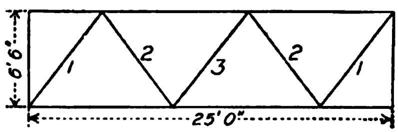


Fig. 114.

the wind load, are found to be ± 4970 and ± 3830 pounds, respectively. The length of a lateral is 90 inches, and hence the least radius of gyration must not be less than $90/120 = 0.75$. This requires the use of $4'' \times 4''$ angles, and with a minimum thickness of $\frac{3}{8}''$ the area is considerably in excess of that required for the stresses, including those due to eccentric connections.

The final estimate of the weight of the stringer is now computed with the aid of the tables in a handbook.

	Pounds.
4 flange angles, $6'' \times 6'' \times \frac{5}{8}'' \times 24'\ 11\frac{1}{2}''$, @ 24.2 lbs.	2416
1 web plate, $42'' \times \frac{1}{2}'' \times 24'\ 11\frac{1}{2}''$, @ 71.4 lbs.	1782
4 connecting angles, $6'' \times 6'' \times \frac{1}{2}'' \times 3'\ 4\frac{3}{8}''$, @ 19.6 lbs. . .	264
4 fillers, $9 \times \frac{5}{8}'' \times 2'\ 5\frac{1}{8}''$, @ 19.13 lbs.	186
Half lateral system:	
$2\frac{1}{2}$ lateral angles, $4'' \times 4'' \times \frac{3}{8}'' \times 7'\ 4''$, @ 9.8 lbs.	180
2 connecting plates, $8'' \times \frac{3}{8}'' \times 1'\ 8''$, @ 10.2 lbs.	34
1 connecting plate, $8'' \times \frac{3}{8}'' \times 11''$ (corner clipped)	8
238 pairs of rivet heads, @ 0.369 lb.	88
Total	4958

In addition to their own weight and that of their lateral system, the stringers support a part of the weight of the lower laterals of the bridge and of their connections to the stringers. The assumed weight of 5100 pounds exceeds that just obtained by a sufficient amount to avoid the necessity of revising the moments and shears due to the dead load. It should also be

remembered that the end connections are really supported by the floor beam, and their weight therefore causes no stresses in the stringers.

The weight of one stringer and one-half of the lateral system is distributed as follows:

	Weight in Pounds.	Percentage of Total Weight.	
Flanges	2416	48.7	84.6
Web plate	1782	35.9	
End connections with floor beams	450	9.1	15.4
Half lateral system	222	4.5	
Rivet heads	88	1.8	
	4958	100.0	100.0

This table also shows that the specification in regard to the depth of the stringer is practically satisfied, for the weight of the flanges is only 184 pounds greater than that of the web, including the end connections. The percentages of these two items are 48.7 and 45 respectively. This indicates that theoretically the total weight is near the minimum. As it seemed probable that a depth of 44 inches might give better results, another design was made which gave the following weights: Flanges, 2187 pounds; web plate, 2098 pounds; end connections, 459 pounds; rivet heads, 78 pounds; lateral bracing, the same as before; and the total, 5055 pounds. The web had to be increased in thickness to $\frac{9}{16}''$ in order to avoid the use of stiffeners, and the flange angles were reduced in thickness to $\frac{9}{16}''$. In this case the weight of the flanges is less than that of the web, and hence the depth of 42″ gives practically the minimum material.

The above analysis of the weight of a stringer is also useful in checking the assumed weight after but few preliminary com-

putations are made to determine the web and flange section. In an example published in the first edition of this part of the text-book, in which flange cover plates and web stiffeners were employed, the live load being only about three-fifths as heavy, and the economic depth 38 inches, the combined weight of the flanges and web plate was found to be 77.6 percent of the entire weight. This indicates that even in extreme cases the variation in this percentage is not very large.

ART. 86. FLOOR BEAMS.

SPECIFICATION. — The effective length shall be the perpendicular distance between the central planes of trusses. All spans shall have end floor beams riveted rigidly to the trusses, to support the stringers. The intermediate floor beams in through bridges are to be riveted between the posts. For unit stresses see Art. 83. Most of the specifications relating to stringers apply also to floor beams.

For convenience in erection, bracket angles are riveted to the lower flange of the floor beam or to the web just above the flange, on which to support the stringers until their end-connecting angles are riveted to the floor-beam webs.

Assuming that the vertical legs of the flange angles do not exceed 4 inches, it is found that a depth of 54 inches will bring the top of the cross-ties between 2 and 3 inches higher than the top of the floor beam, provided the bracket angle has a 5-inch vertical leg and is riveted to the lower flange angles.

The floor beam carries, in addition to its own weight, two concentrated loads 3′ 3″ from its center, each load consisting of the maximum sum of the adjacent reactions of the stringers on both sides. This sum includes the weight of one stringer, and of the track which it supports, and the corresponding live load. Using the same values as in the preceding article, the dead load of one stringer is $5500 + 5100 = 10\,600$ pounds. The equivalent uniform live load must be taken for a span of two panel lengths, or 50 feet, and by means of WADDELL's diagram it is found to be

8000 pounds per linear foot per track. The live load at each stringer connection is therefore $4000 \times 25 = 100\,000$ pounds. The allowance for impact is 85 700 pounds, and the total load 196 300 pounds. The approximate weight of the floor beam is assumed to be 4200 pounds.

The maximum vertical shear is 198 400 pounds, and for the allowable shearing stress of 10 000 pounds per square inch, the net section of the web is 19.84 square inches. A thickness of $\frac{1}{2}''$ will allow for about 14 rivet holes in the net section of the splice, which is probably sufficient. Assuming one-eighth of the gross section as the equivalent flange area of the web plate, its value is 3.37 square inches.

It is desirable to determine the rivet pitch approximately before the flange angles are definitely selected, in order to know whether the rivets can be placed in a single row. The effective span of the floor beam is 17 feet, the distance between the centers of trusses. The bending moment due to the two concentrated loads is 1 030 600 pound-feet, and that due to the weight of the floor beam is 8900 pound-feet, making the total 1 039 500 pound-feet. The moment diagram is shown in Fig. 115, the moment due to dead load being laid off below the axis. Assuming an effective depth of 53 inches, and using the specified unit stress, the required net area of the lower flange is

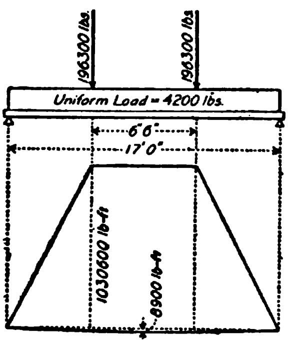


Fig. 115.

$$\frac{1\,039\,500 \times 12}{53 \times 15\,000} - 3.37 = 12.32 \text{ square inches.}$$

The approximate increment of flange stress per linear inch between the end and the stringer connection is $12.32 \times 15\,000/63$

= 2934 pounds. The pitch is, therefore, about 9630/2934 = 3.28 inches, the bearing of a $\frac{7}{8}''$ rivet in the $\frac{1}{2}''$ web plate being 9630 pounds. This indicates that only one row of rivets is needed to connect the flange angles to the web.

Let the following composition of the flange be taken, which furnishes a net area of 12.30 square inches:

2 angles, $5'' \times 4'' \times \frac{9}{16}''$,	$2\,(4.75 - 1.13) =$	7.24 square inches.
1 cover plate, $11'' \times \frac{9}{16}''$,	$(6.19 - 1.13) =$	5.06
		12.30

The center of gravity of the solid section of the upper flange is $0.55''$ below the backs of the angles, and that of the net section of the lower flange is $0.60''$ above the backs of the angles, the rivet holes in the vertical legs of the angles being also deducted, since they are less than $2\frac{1}{2}$ inches from the section through the rivet holes in the horizontal legs of the angles and in the cover plates. Placing the backs of the angles $\frac{1}{8}''$ beyond the edges of the web plate, the effective depth is $54 + 0.25 - 0.60 - 0.55 = 53.1$ inches. This value reduces the required net area of the flanges to $15.66 - 3.37 = 12.29$ square inches. The above composition of the flanges may therefore be adopted, provided it is found later that the equivalent flange area of the web is not less than that assumed.

As shown in Fig. 116, the web plate will be spliced in order to allow the bottom of the floor beam to be on the same level as that of the post, that the connecting plates of the lateral system may be riveted to both, and also to allow the end web plate to be extended above the upper flange for the connection to the post. The method of designing this splice is the same as that explained in Art. 56. The rivets in the lower half of the inner row of rivets are located at the following distances from the neutral surface, all expressed in inches: 0, 3.5, 6.5, 9.5, 12.5, 15.5, 18.5, 21.5, and 24.625. The last distance is that of the

rivet in the flange angles. The squares of these numbers are: 0, 12.25, 42.25, 90.25, 156.25, 240.25, 342.25, 462.25, and 606.37. Their sum is 1952.1 inches². The resisting moment of the lower half of the solid web plate is 1 822 500 pound-inches, and the moment deducted by the rivet holes in this row is $7500 \times 1952.1/27.125 = 539\,750$ pound-inches, 7500 pounds being the tensile strength deducted by one rivet hole at the distance of the outer fiber, or 27.125 inches, from the neutral surface. If the outer row of rivets had the same number of rivets as the inner one, the resisting moment of the net section would be

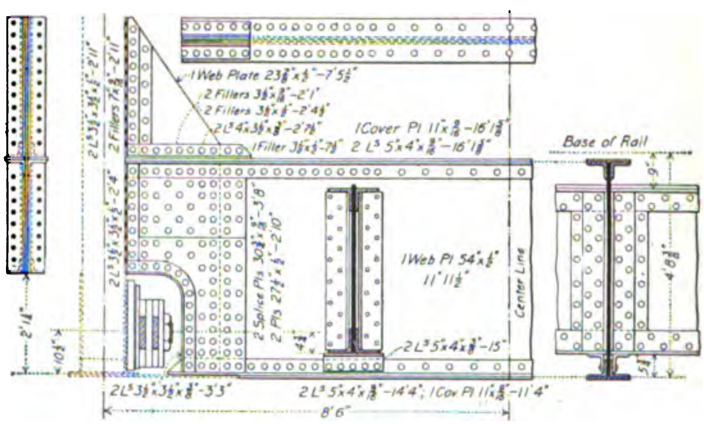


Fig. 116.

1 282 750 pound-inches. It must be a little larger than this because the net section of the web allows only 14 rivet holes. The sum of the moments of the bearing values of the rivets in the web plate is $9630 \times 1952.1/24.625 = 763\,400$ pound-inches, 9630 pounds being the bearing value of the rivet which is 24.625 inches from the neutral surface. This leaves a moment to be resisted by the rivets in the outer row of 519 350 pound-inches, and requires Σy^2 (see Art. 56) to be 1328 inches². Let the rivets be omitted whose distances are 3.5, 9.5, and 15.5 from the neutral surface, making Σy^2 equal to 1609.3 inches².

On revision the reduction of resisting moment due to the rivet holes in the outer row is $7500 \times 1609.3/27.125 = 445\,000$ pound-inches, making the net resisting moment of the web plate 1 377 500 pound-inches. The required moment of the bearing values of the rivets in the outer row is then 614 000 pound-inches and $\Sigma y^2 = 1570$ inches². This indicates that the strength of the splice is practically equal to that of the net section of the web plate, the rivets in the flange angles being included in both rows. That is allowable, because in this case the flanges have a large excess of section at the web splice, and the spliced plates extend clear to the left end of the beam. As the right-hand row contains only 11 rivets, the web has sufficient area to resist the shear.

The net section of the web plate is $1\,377\,500/1\,822\,500 = 0.756$ or 75.6 percent of the strength of the gross section, and hence one-sixth of this, or 12.6 per cent of the gross sectional area of the web plate, equals its equivalent flange area. This gives 3.40 square inches, which agrees almost exactly with the assumption previously made.

The upper cover plate is extended to the post, it being slotted at the end to allow the projecting web plate to pass through. The lower cover is extended as far as the connecting plate for the lateral system permits. The rivet pitch may next be revised. The increment of flange stress per linear inch is

$$\frac{12.30}{15.70} \cdot \frac{198\,400}{53.1} = 2927 \text{ pounds,}$$

and the pitch is $9630/2927 = 3.29$ inches. It may therefore be made $3\frac{1}{4}''$ in the spaces outside of the stringers, but is preferably reduced to 3 inches. In the space between the stringers the pitch is made 6 inches, the maximum allowed.

In the design of the stringer it was found that 30 field rivets are required to connect the two end angles of each stringer to

the floor-beam web. It remains to determine how many are required to carry the whole load of 196 300 pounds at each pair of stringer connections. Since the rivets are in double shear, their bearing in the web of the floor beam will govern, and the number is 196 300/9630 = 21 shop rivets or 26 field rivets. The number previously found will therefore be used.

The equivalent flange area of the web which must be used in the final determination of the net flange area is that of a section taken through one of the rows of rivets in the stringer connection. For a section through the outer row in which the rivets are farther from the neutral surface, their distances being 3.5, 8.5, 13.5, 18.5, and $24\frac{5}{8}$ inches respectively, the resisting moment is 81.6 percent of that of the solid plate, and hence its equivalent flange area is 13.6 percent of 27 square inches, or 3.67 square inches. The composition of the flange therefore requires no revision.

In the end connection of the floor beam, the number of shop rivets required to transmit the shear from the web into the fillers is 198 400/9630 = 21, and the number needed to carry it into the angles is 198 400/13 220 = 15. To connect the end angles to the post requires 198 400/6610 = 30 shop rivets, but as field rivets are used their number must be increased to 38.

The splice plates are extended to the end of the beam in order to act also as fillers under the end angles and to transfer the stresses to these angles in the most direct manner. An additional pair of plates is placed on the sides of the floor beam around the corner cut, in order to strengthen the web around the cut and also to serve as fillers for the curved flange angles.

The following table gives the weight of one floor beam:

	Pounds.
2 flange angles, $5'' \times 4'' \times \frac{9}{16}'' \times 16' 1\frac{5}{8}''$, @ 16.2 lbs. . . .	523
2 flange angles, $5'' \times 4'' \times \frac{9}{16}'' \times 14' 4''$, @ 16.2 lbs.	464
1 cover plate, $11'' \times \frac{9}{16}'' \times 16' 1\frac{5}{8}''$, @ 21.02 lbs.	339

	Pounds
1 cover plate, $11'' \times \frac{9}{16}'' \times 11'\ 4''$, @ 21.02 lbs.	238
1 web plate, $54'' \times \frac{1}{2}'' \times 11'\ 11\frac{1}{2}''$, @ 91.84 lbs.	1022
2 web plates, $23\frac{7}{8}'' \times \frac{1}{2}'' \times 7'\ 5\frac{1}{2}''$, @ 40.59 lbs. (less 223 lbs.) .	383
4 splice plates, $30\frac{3}{4}'' \times \frac{9}{16}'' \times 3'\ 8''$, @ 58.83 lbs. (less 226 lbs.) .	637
4 filler plates, $27'' \times \frac{1}{2}'' \times 2'\ 10''$, @ 45.92 lbs. (less 201 lbs.) .	316
4 connecting angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}'' \times 2'\ 4''$, @ 11.1 lbs. . .	104
4 connecting angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}' \times 2'\ 11''$, @ 11.1 lbs. . .	130
4 fillers, $7'' \times \frac{9}{16}'' \times 2'\ 11''$, @ 13.39 lbs.	156
4 angles, $4'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 2'\ 7\frac{1}{2}''$, @ 9.1 lbs.	96
4 fillers, $3\frac{1}{2}'' \times \frac{9}{16}'' \times 2'\ 1''$, @ 6.7 lbs.	56
4 fillers, $3\frac{1}{2}'' \times \frac{1}{2}'' \times 2'\ 4\frac{1}{2}''$, @ 5.95 lbs.	56
2 fillers, $3\frac{1}{2}'' \times \frac{1}{2}'' \times 7\frac{1}{2}''$, @ 5.95 lbs.	7
4 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 3'\ 3''$, @ 8.5 lbs.	111
4 bracket angles, $5'' \times 4'' \times \frac{3}{8}'' \times 1'\ 3''$, @ 11.0 lbs.	55
596 pairs of rivet heads @ 0.369 lb.	220
Total	4915

The weight is distributed as follows:

	Weight in Pounds.	Per-cent.
Flanges	1564	31.8
Web plates	1405	28.6
Splices, end connections, etc.	1726	35.1
Rivet heads	220	4.5
Total	4915	100.0

This analysis shows how much material is required in order to bring the bottom of the floor beam even with the bottom of the post and to clear the chord bars. As shown in Fig. 116, there are five plates riveted together to form the web around the corner cut. As so large a percentage of the weight of the floor beam is concentrated near the ends, no revision is needed on account of the difference between the assumed and the actual weights.

The design of the end floor beam is left as an exercise for the student. See Plate IV for an example showing the form and details of its end connections. The design of the end floor

beam cannot be completed until after that of the end posts to which it is connected.

ART. 87. STRESSES IN TRUSSES.

The following data and dimensions are tabulated below for convenient reference:

Span, center to center of end pins	175′ 0″
Depth, between centers of chords	31′ 0″
Width, between centers of trusses	17′ 0″
Number of panels	7
Panel length	25′ 0″
Length of end post, center to center of pins, 39.825′ =	39′ 9.9″

$$\theta = 38^\circ\ 53' \qquad \theta' = 55^\circ\ 47'$$

$$\tan\theta = 0.8065 \qquad \tan\theta' = 1.4706$$

$$\sec\theta = 1.2847 \qquad \sec\theta' = 1.7784$$

The angle which the diagonals of the truss make with the vertical is θ, and that of the diagonals of the lateral systems with the struts is θ'. The opposite truss has the same letters as the one in Fig. 117, except that the letters are primed.

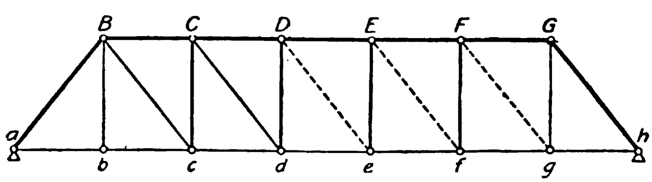


Fig. 117.

The weight per linear foot of the track is 440 pounds, that of the stringers and floor beams is 594 pounds, and that of the trusses and lateral systems is assumed to be 1336 pounds, making a total of 2400 pounds. The dead panel load per truss is 30 000 pounds; one-third of which will be applied at

the upper panel points, and the remainder at the lower panel points.

The equivalent uniform live load is shown by the diagram to be 6325 pounds per linear foot per track, which gives a panel load per truss of 79 100 pounds. The panel load for the suspender *Bb*, however, is equal to the floor beam reaction, or 100 000 pounds.

The specified wind load will be considered as equally divided between the windward and leeward panel points of each lateral system. The panel load for the upper system is 1875 pounds. The panel loads for the lower system are 1875 and 5625 pounds respectively for the static and moving wind loads. The panel load for the overturning moment of the wind pressure on the train is 7080 pounds, the distance from the base of the rail to the lower lateral system being estimated as 4.7 feet.

The stresses in the trusses and lateral bracing were computed according to the methods of Part I, and are given in the following tables, their values being expressed in kips, one kip being equal to 1000 pounds:

	END POST.	UPPER CHORD.		LOWER CHORD.		
	aB	*BC*	*DE*	*bc*	*cd*	*de*
Dead load	−115.6	−121.0	−145.2	+ 72.6	+121.0	+145.2
Live load	−304.8	−319.1	−382.8	+191.4	+319.1	+382.8
Impact allowance	−192.6	−201.6	−241.9	+121.0	+201.6	+241.9
Wind overturning:						
On truss, east	− 26.4	− 16.5	− 16.5	+ 16.5	+ 16.5	+ 16.5
On truss, west	+ 26.4	+ 16.5	+ 16.5	− 16.5	− 16.5	− 16.5
On train, east	− 27.3	− 28.6	− 34.3	+ 17.1	+ 28.6	+ 34.3
On train, west	+ 27.3	+ 28.6	+ 34.3	− 17.1	− 28.6	− 34.3
Wind on truss, east		0	+ 16.5	+ 16.5	+ 27.6	+ 33.1
Wind on truss, west		− 11.0	− 16.5	− 27.6	− 33.1	− 33.1
Wind on train, east				+ 49.6	+ 82.7	+ 99.3
Wind on train, west				− 82.7	− 99.3	− 99.3

	Diagonals.					Verticals.		
	Bc	*Cd*	*De*	*Ef*	*Fg*	*Bb*	*Cc*	*Dd*
Dead load	+ 77.1	+ 38.5	0	−38.5	−77.1	+ 20.0	− 40.0	−10.0
Live load	+217.8	+145.2	+87.1	+43.6	+14.5	+100.0	−113.0	−67.8
Impact allowance	+145.2	+102.5	+65.4	+34.8	+12.5	+ 85.7	− 79.8	−50.9
Wind overturning:								
On train, east	+ 19.5	+ 13.0	+ 7.8	+ 3.9	+ 1.3	+ 7.1	− 10.1	− 6.1
On train, west	− 19.5	− 13.0	− 7.8	− 3.9	− 1.3	− 7.1	+ 10.1	+ 6.1

The stresses in CD are the same as those in DE, except for wind on truss, west, which is + 11.0 instead of + 16.6 kips. The stresses in ab are the same as those in bc, except those in the last four lines of the table, for which the following values are to be substituted: 0, − 16.5, 0, and − 49.6 kips.

The wind stresses in the upper laterals, expressed in kips, are: BC', + 13.3; CD', + 6.7; DE', 0; while those in the struts are: BB', − 3.8; CC', − 5.6; DD', − 1.9. The wind stresses due to both static and moving wind loads in the lower laterals are: ab', + 80.0; bc', + 56.2; cd', + 35.3; de', + 17.2; while those in the struts are: aa', − 22.5; bb', − 37.5; cc', − 24.1; and dd', − 12.3.

Art. 88. Sections of Intermediate Posts.

Specification. — The effective length shall be the greatest length between points of the axis that are rigidly held in the direction in which the strength is being considered. The least width of posts in pin-connected trusses shall be limited to 10 inches.

Let it be required to design the section of the post Cc. Neglecting the wind stresses, which are relatively too small to affect the result according to the specified unit stresses, the total stress to be considered is 232 800 pounds. A trial shows that 15-inch channels are required. Taking the radius of gyration about the neutral axis perpendicular to the web of the channels

as 5.43 inches, the value of l/r equals $31 \times 12/5.43 = 68.5$. By means of a table of unit stresses for values of l/r in the specified column formula ranging from 10 to 120, the corresponding average unit stress is found to be 11 110 pounds per square inch, which requires a sectional area of $232\ 800/11\ 110 = 20.96$ square inches. Referring to the handbook, it is found that two 15-inch channels, each weighing 40 pounds per linear foot, will give an area of 23.52 square inches. As the radius of gyration of these channels agrees with the value assumed, no revision is needed, and these channels are accordingly adopted. The flanges will be turned inward so as to avoid cutting them near the pin connections. It remains to determine the distance back to back of channels, in order that the moments of inertia about the two rectangular axes through the center of the section may be equal, thus rendering the column of equal strength against lateral flexure in these two directions, provided it may be equally free to bend in either direction. Let x be this distance, then by the application of the principles of moment of inertia (see Text-book on Mechanics of Materials, Arts. 43 and 76) there results the equation

$$2 \times 347.5 = 2\,[9.39 + 11.76\,(\tfrac{1}{2}x - 0.783)^2]$$

whose solution gives $x = 12.29$ inches. In this equation 347.5 inches^4 is the moment of inertia of one channel about the axis AB (Fig. 118), 9.39 inches^4 is the moment of inertia about the axis CD, and 0.783 inch is the distance from the axis CD to the back of the channel, all these elements being given in the handbook.

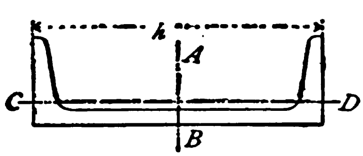


Fig. 118.

Some of the handbooks also give the distances back to back of channels, in order to make the radii of gyration equal.

In a similar manner it is found that two 12-inch 25-pound channels are needed for the post *Dd*, the radius of gyration being 4.43 inches, the average unit compression 9850 pounds per square inch, the required sectional area 13.06 square inches, and the area furnished by the channels 14.70 square inches. The distance back to back of channels for equal radii of gyration is 10.07 inches. The channels in the posts are placed with their webs parallel to the plane of the truss.

Since all the post channels must have the same spacing, in order that the lengths of the floor beams may be the same, and as it is also important to make that distance as small as possible, because the width of the upper chord depends upon it, a computation may be made to see what the spacing of the channels in *Cc* must be, so that its area shall just equal that required. The value is found to be 10.31 inches. A uniform spacing of $10\frac{3}{8}$ inches will accordingly be adopted.

ART. 89. SECTIONS OF DIAGONALS AND SUSPENDER.

SPECIFICATION. — Counter-stresses must be provided for wherever caused by the increased live load (see Art. 83); and in case of reversal of stress the member must be designed to resist such reversal. The use of more than a single system of cancellation in bridges shall be confined entirely to lateral systems and sway bracing, except that in the middle panels of trusses two rigid diagonals connected at their intersection may, for appearance, be employed, provided that either diagonal have sufficient strength to carry the entire shear in either tension or compression, and that the adjacent vertical posts be figured accordingly. All through spans shall have stiff end vertical suspenders.

Since the minimum stress in *Bc* is a tension of 48 800 pounds, it may be composed of one or more pairs of eye-bars. The wind stress may be neglected in designing the member according to the specifications. For the unit tensile stress of 15 000 pounds per square inch, the sectional area must be 440 100/15 000 = 29.34 square inches. Two eye-bars, $8'' \times 1\frac{7}{8}''$, provide an area

of 30 square inches. For a depth of 7 inches the thickness would have to be $2\frac{1}{8}$ inches, which is not so desirable. The thickness of eye-bars ranges in practice from one-fourth to one-seventh of their depth or width. These limits are exceeded only in rare instances.

In accordance with the specifications no counter-tie is allowed in the third panel, and hence the diagonal *Cd* must be designed to take also a compression equal in magnitude to the tension given in the table for *Ef*, in Art. 87. Let two 15-inch 50-pound channels be tried. The required net area for tension is $286\,200/15\,000 = 19.08$ square inches. The radius of gyration $r = 5.23$ inches, the length $l = 477.9$ inches, $l/r = 91.4$, the allowable average unit compression $p = 9260$ pounds per square inch, and the required area for compression is $39\,900/9260 = 4.31$ square inches. The net section must therefore exceed 19.08 square inches, while the gross section must not be less than 23.39 square inches. The gross section of these channels is 29.42 square inches, their web thickness is 0.72 inch, and the grip of the rivets in their flanges is 0.625 inch. Except near the ends the only rivet holes are those in the flanges needed for the lacing. The end connections require pin plates to be riveted to the webs, and in channels of this size four lines of rivets will be used. As parts of the stay plates and pin plates will be opposite each other, it will be necessary to deduct the area of four rivet holes in the web and two in the flanges of each channel. This leaves a net section of $29.42 - 5.76 - 2.50 = 21.16$ square inches. The 45-pound channels will not do, as their net area falls below that required for the tension alone.

It remains to test the section by the provision of the specifications which is intended to make allowance for future increase of the live load. The area required for tension and compression under the increased live-load stresses are 17.80 and 6.54 square

inches respectively. The 50-pound channels may therefore be adopted.

According to the specifications each of the diagonals in the middle panel must be designed to take the entire stress in either tension or compression. The net area in tension is $152\,600/15\,000 = 10.17$ square inches. As the diagonals are riveted to common connecting plates at the middle, the length to be used in the column formula is one-half of the total length of *De*, or 239 inches. Let two pairs of angles, $5'' \times 3\frac{1}{2}'' \times \frac{9}{16}''$, laced together so as to form a section like Fig. 86 be tried. The lacing will be $\frac{3}{8}''$ thick, and hence the space between the backs of the angles must be $\frac{3}{4}''$. The radius of gyration $r = 2.60$ inches, $l/r = 91.9$, $p = 9210$ pounds per square inch, and the required gross area is $152\,600/9210 = 16.57$ square inches. The angles have a gross area of 17.88 square inches, and provide more than the needed net area when a rivet hole is deducted from both legs of the angles.

The suspender, *Bb*, will be designed as a stiff member for the reasons given in Art. 76, its composition being made like that of the intermediate posts. Its required net sectional area is $205\,700/15\,000 = 13.71$ square inches. Two 12 inch 30-pound channels will be selected, as they will furnish 13.80 square inches, after deducting two rivet holes in both the web and flanges of each channel. The net section is at the connection to the upper pin plates, and by staggering the rivets in the three rows connecting the webs to the pin plates it may be arranged that the flange rivets through the stay plates shall be in the same cross-section as the middle rivets in the webs, thus increasing the available section to 14.74 square inches.

The insufficient provision frequently made for counter-stresses in railroad bridges is discussed in a paper by H. S. PRICHARD, in Transactions American Society of Civil Engineers, vol. 42, page 547, Dec., 1899.

ART. 90. LOWER CHORD SECTIONS.

SPECIFICATION. — For single-track spans the two panels of the lower chord, at each end, shall preferably be made rigid members.

If the wind stresses be neglected, the required net area for the lower chord member *cd* is 641 700/15 000 = 42.78 square inches, while if they be included, the net area is 797 100/19 000 = 41.95 square inches. The larger area must be taken. Four taken. Four eye-bars $8'' \times 1\frac{3}{8}''$ give 44 square inches, and are therefore chosen.

In the case of *de* the greater sectional area is obtained by omitting the wind stresses, its value being 769 900/15 000 = 51.33 square inches. Four eye-bars, $8'' \times 1\frac{5}{8}''$, are needed whose combined sectional area is 52 square inches.

The stresses which govern the design of *ab* and *bc* are the same, and hence a single member may be extended from *a* to *c*. The required net area is 385 000/15 000 = 25.67 square inches. Let *ac* be composed of two built channels, like Fig. 90. Since the eye-bar heads of the 8-inch eye-bars are 17 inches deep according to the handbook, let the web plates be made 18 inches deep so as to avoid cutting the angles in order to pass the eye-bar heads at *c*. Selecting 2 web plates $18'' \times \frac{1}{2}''$, and 4 angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, the rivets in the end pin plates can be so arranged as not to deduct more than 3 rivet holes in each web plate and one in each angle, giving a net area of 26 square inches.

The specifications also require that, if the unit stress due to the weight of a member is greater than 10 percent of the safe value allowed, the sectional area must be increased. According to the first method given in Mechanics of Materials, Art. 103, the stress in an eye-bar 8 inches deep and 25 feet long, due to its own weight, is found to be 1187 pounds per square inch, which is less than the above limit.

The formula is
$$S_1 = \frac{Mc}{I + \frac{nPl^2}{mE}},$$

in which M is the bending moment of the flexural forces, c the distance from the neutral surface to the outer fiber in which the tension S_1 occurs, I the moment of inertia of the cross-section, n and m numbers depending upon the arrangement of the ends and the kind of loading, P the longitudinal tensile force, l the length of the member, and E the coefficient of elasticity of the material. The flexure of the eye-bar under its own weight corresponds to that of a simple beam uniformly loaded, and hence $m/n = 9.6$. The same result is obtained for any thickness of bar, and hence a single determination only is required for all bars of the same depth and length. The second and more accurate method given in the same article gives a stress of 1176 pounds per square inch. The value of E for soft steel is taken as 26 000 000 pounds per square inch.

The results of some experiments on the relative strength of eye-bars and built members of various forms of section are given in a paper entitled 'Recent Tests of Bridge Members,' by J. E. GREINER, in Transactions American Society of Civil Engineers, vol. 38, page 41, Dec., 1897.

ART. 91. DIAMETERS OF PINS.

SPECIFICATION. — The stress in the outer fibers of pins shall not exceed 25 000 pounds per square inch, the points of application of the stresses in the connecting members being taken at the centers of bearings. In designing all pin-connected work ample clearance for packing must be provided, and ample room must be left for assembling members in confined spaces. Lower chords are to be packed as closely as possible, and in such a manner as to produce the least bending moments on the pins, but adjacent eye-bars in the same panel must never have less than a one-half-inch space between them, in order to facilitate painting. The various members attached to any pin must be packed as closely as practicable, and all interior vacant spaces must be filled with steel fillers, where their omission would permit of the motion of any

member on the pin. All bars are to lie in planes as nearly as possible parallel to the central plane of the truss, no divergence exceeding one-eighth of an inch to the foot being permitted.

In order to find the bending moment in the pin at *d* it is necessary to determine provisionally the thickness of the pin plates for the post *Dd* and the diagonals *Cd* and *Ed*. As the specifications do not permit countersunk rivet heads in metal less than $\frac{7}{16}''$ thick, one pin plate of this thickness will be placed on each side of the web of each channel in the post. These plates and the webs will furnish sufficient bearing area for a pin whose diameter is not less than $5\frac{1}{2}$ inches. The pin cannot be less than $8 \times 15\,000/22\,000 = 5.46$ inches for 8-inch eye-bars, in order to provide adequate bearing area for the eye-bars. Assuming that the pin will not exceed 6 inches in diameter, it is estimated that a $\frac{9}{16}''$ pin plate will be required on each side of the webs of the channels composing *Cd*, in order that the net sectional area at the pin hole shall not be less than 40 percent in excess of that of the net section elsewhere. The diagonal *Ed* is connected to the pin by means of pin plates only whose combined thickness on each side is 1 inch, the angles being cut off so as not to interfere with the post.

The horizontal forces in the chord bars and diagonals produce flexure in the pin in a horizontal plane, the bending moment being designated by M_h, while the vertical forces in the diagonal and post produce a moment M_v, their resultant being

$$M = \sqrt{M_h^2 + M_v^2}.$$

In order to reduce M_v, the bearings of the diagonal are placed outside of and next to those of the post. The eye-bars in the chord alternate in direction so as to allow space between adjacent bars of the same panel for painting. To reduce M_h, one of the smaller bars is put on the outside. The arrangement, or packing, of the eye-bars, etc., on one side is indicated in Fig. 119.

A clearance of $\frac{1}{16}''$ is allowed between eye-bars, and $\frac{1}{8}''$ where one or both of the adjacent members contain countersunk rivet heads. The stresses marked on the chord bars are the safe values for their actual sections. The stress in *Cd* is that required to make the algebraic sum of all the horizontal forces acting on the pin equal to zero. The straight sides of the equilibrium polygon are drawn by considering the stresses applied at the centers of the respective bearings, as specified,

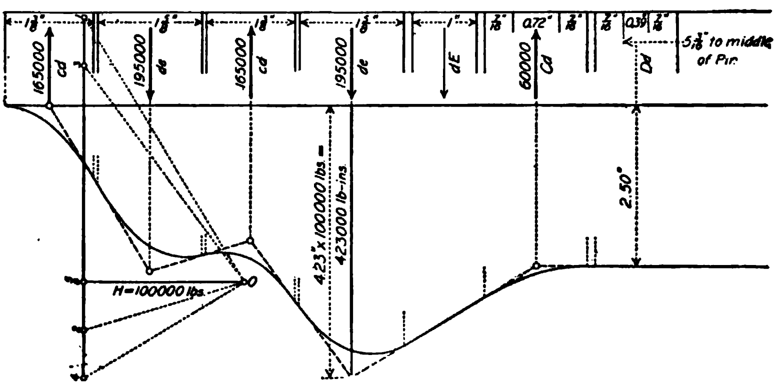


Fig. 119.

while the curved lines give the form of the diagram when the stresses are regarded as uniformly distributed on the bearings. The curves are parabolas, the points of tangency lying in the ordinates through the sides of the eye-bars, and other members (Part II, Art. 10). The original diagram was drawn full size for the linear dimensions, and the pole distance made equal to 100 000 pounds. The ordinate at the post bearing, where M_v is a maximum, measured 2.50 inches, making

$$M_h = 2.50 \times 100\,000 = 250\,000 \text{ pound-inches},$$

$$M_v = 74\,400 \times 1.49 = 110\,900 \text{ pound-inches},$$

74 400 pounds being the vertical component of the stress in *Cd* which corresponds to the horizontal component of 60 000

pounds, while 1.49 inches is the distance between the centers of bearing of the diagonal and post. Both horizontal and vertical moments are uniform between the two sides of the post, as there are no forces acting on the pin within those limits. The resultant M is 273 400 pound-inches, but since this value is less than M_h at the inner bar of dc where there is no M_v, the maximum moment is 423 000 pound-inches. According to the handbook this requires a pin $5\frac{5}{8}$ inches in diameter, whose resisting moment is 436 800 pound-inches.

In a similar manner the bending moments are found in the pin at c, it being estimated that two pin plates are required for each half of the stiff chord member bc, one $\frac{5}{8}''$ and the other $\frac{1}{2}''$ in thickness. The greatest value of M_h is 446 000 pound-inches at the post bearing, while M_v is 295 000 pound-inches, making $M = 529\ 500$ pound-inches. A 6-inch pin whose resisting moment is 530 200 pound-inches will therefore be required.

If the outer eye-bars in cd be reduced in thickness to $1\frac{1}{8}$ inches and the inner ones increased to $1\frac{5}{8}$ inches, the other members remaining the same, M_h is reduced to 340 000 pound-inches, and M to 443 500 pound-inches, thus requiring a pin $5\frac{3}{4}$ inches in diameter. This change makes a still larger reduction in the moments on the pin at d, and indicates the method by which the diameter of the pin may be brought within given limits when desired.

The bending moments on the pins at a and B need not usually be found, as they are smaller than those at c and d. The pins at those panel points will, however, be made the same size, so as to reduce the total thickness of pin plates otherwise required. Pins 6 inches in diameter will then be adopted at a, c, d, and B. Those at C and D will be designed after the upper chord sections are determined.

ART. 92. UPPER CHORD SECTIONS.

SPECIFICATION. — In members subject to compression, rivets shall be so spaced that they shall not be farther apart in the direction of the stress than sixteen times the thickness of the thinnest external plate connected, and not more than fifty times that thickness at right angles to the direction of the stress.

The standard diameter of the head of an 8-inch eye-bar when the pin does not exceed 6 inches is 17 inches, according to one of the handbooks. If the eye-bars of *Bc* are placed inside of the upper chord, the web plates must be at least 18 inches deep to provide ample clearance. The width of the chord must also be determined at this panel point. For a single-track bridge of 175 feet span a composition like that shown in Fig. 96 is appropriate. In Art. 88 the distance back to back of the channels in the posts was found to be $10\frac{5}{8}$ inches, and hence those in the suspender will be spaced the same distance. The pin plates on the outside of the suspenders are estimated to be $\frac{11}{16}''$ thick. Considering one side only for simplicity, and allowing $\frac{1}{8}''$ for a clearance next to the pin plate, $1\frac{7}{8}''$ for the thickness of one eye-bar in *Bc*, $\frac{1}{4}''$ for another clearance, $\frac{5}{8}''$ for a jaw plate, and $\frac{1}{2}''$ for a pin plate, both connected to the inside of the upper chord web, and $\frac{1}{2}''$ for the thickness of web plate, the distance from the center of the suspender to the outside of the chord web plate, or to the backs of the angles, is found to be $9\frac{1}{2}$ inches, and if the angles be $3\frac{1}{2}$ inches wide, the cover plate must be 26 inches wide. The lines of rivets connecting it to the angles are 23 inches apart, and hence the cover plate must be $\frac{7}{16}''$ thick to satisfy the requirement of the specifications regarding the maximum spacing of rivets at right angles to the direction of the stress in compression members.

Since the specified unit stress involves the radius of gyration, an approximate value must be assumed. A convenient rule

makes the radius of gyration about a horizontal axis equal to four-tenths of the depth out to out. This depth is estimated to be 19.44 inches, making $r = 7.78$ inches, $l/r = 38.6$, $p = 13\,510$ pounds per square inch, and the required sectional area

$$641\,700/13\,510 = 47.50 \text{ square inches.}$$

The composition of the section is as follows:

1 cover plate, $26'' \times \frac{7}{16}''$	11.37 square inches.
4 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	11.48
2 web plates, $18'' \times \frac{7}{16}''$	15.76
2 flats, $5'' \times 1''$	10.00
Total	48.61

Placing the backs of the angles $\frac{1}{8}''$ beyond the edges of the web plates, the center of gravity is found to be $0.21''$ above the center of the web plates. The pin may accordingly be placed with its axis passing through the centers of the web plates, for it is found that an eccentricity of $0.24''$ would just make the negative moment due to the maximum direct compression equal in magnitude to the positive bending moment produced by the weight of the chord.

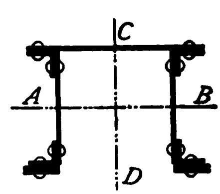


Fig. 120.

The following computation gives the moment of inertia with reference to the neutral axis AB in Fig. 120:

1 cover plate, $\frac{1}{12} \times 26(\frac{7}{16})^3$	=	0.2
$11.37(0.22 + 0.125 + 9.0)^2$.	=	992.9
4 angles, 4×3.26	=	13.0
$11.48(9.125 - 1.04)^2$	=	750.4
2 web plates, $2 \times \frac{1}{12} \times \frac{7}{16} \times 18^3$	=	425.2
2 flats, $2 \times \frac{1}{12} \times 5 \times 1^3$	=	0.8
$10(9.125 + 0.5)^2$	=	926.4
		3108.9
$48.61(0.21)^2$	=	2.1
I	=	3106.8 inches^4.

For convenience the moment of inertia is first computed for a horizontal axis through the centers of the web plates and then reduced to the neutral axis. The radius of gyration is

$$r = \sqrt{\frac{3106.8}{48.61}} = 8.00 \text{ inches},$$

and the revised area is found to be 47.25 square inches.

As the stress in CD is larger than that in BC, and its additional sectional area will be placed in the sides, leaving the cover plate and flats unchanged, its radius of gyration will be a little smaller than that for BC, and hence $r = 7.78$ inches will again be assumed. The approximate sectional area for CD is then $769\,900/13\,510 = 56.99$ square inches. Using the same composition as for BC except to increase the thickness of the web plates to $\frac{11}{16}''$, the area is 57.61 square inches. The moment of inertia is then computed to be 3350.2 inches^4, the radius of gyration 7.71 inches, and the revised required area 57.07 square inches. The use of $\frac{9}{16}''$ angles and $\frac{5}{8}''$ web plates would also satisfy the requirements, but the gross area is then 58.37 square inches. The former composition also has the advantage in not requiring the pin plates at C and D to connect to the angles, but only to the web plates because the entire increment of chord stress is taken by the web plates. The section for DE equals that for CD.

Inspection shows that the moments of inertia around the neutral axis CD in Fig. 120 are respectively greater than those computed for the sections of both chord members, and hence the values of r determined above are the least radii of gyration required in the column formula.

The diameter of the pin at C may now be determined. The pin plates on the post Cc are $\frac{7}{16}''$ thick, and those on the diagonal Cd are approximately $\frac{9}{16}''$ thick, one plate being placed on each side of each channel web. One $\frac{3}{8}''$ pin plate is also needed on

the outside of each web plate of the chord if the pin is not less than 5 inches in diameter. Remembering that the channels in the post are spaced $10\frac{3}{8}''$ back to back, and allowing for a clearance of $\frac{1}{8}''$ between the adjacent pin plates of the posts and diagonals, the distance from the center of one of the chord bearings to that of the diagonal is 2.796 inches, and from the latter to the center of the adjacent post bearing is 1.526 inches. Since the chord section is continuous past the pin, the maximum bending moment on the pin occurs when the stress in the diagonal is a maximum. The full strength of the net section of *Cd* is $21.16 \times 15\,000 = 317\,400$ pounds. Its horizontal component of 199 200 pounds equals the corresponding increment of chord stress, and its vertical component of 241 400 pounds is the corresponding stress in the post. $M_h = 99\,600 \times 2.796 = 278\,500$ pound-inches, and $M_v = 120\,700 \times 1.526 = 184\,200$ pound-inches, their resultant M being 333 900 pound-inches. This requires a pin whose diameter is $5\frac{1}{4}$ inches. The same size will be adopted for that at the panel point D.

In case it be desired to reduce the thickness of the flats, an alternative section may be designed by increasing the horizontal legs of the angles to a width of 5 inches. Since the gage of the 5-inch leg for a single row of rivets is 3 inches, a flat 6 inches wide will not extend beyond the back of the angle. This width requires a thickness of $\frac{3}{4}$ inch for the flats and increases the total chord section by 0.32 square inch, but reduces the eccentricity of the neutral axis to 0.18 inch. If, however, the flats be taken 7 inches wide, their required thickness is $\frac{5}{8}$ inch, the corresponding increase in chord section being only 0.08 square inch, while the eccentricity is increased to 0.24 inch.

ART. 93. SECTION OF INCLINED END POST.

SPECIFICATION. — The inclined end post must be so proportioned that the algebraic sum of the stresses per square inch resulting from the direct com-

pression and the maximum bending moment due to the wind pressure shall not exceed 19,000 pounds per square inch. Every column that acts as a beam also must have solid webs at right angles to each other, as no reliance shall be placed on lacing to carry a transverse load down the column.

The maximum direct compression in the end post aB is not quite as large as that in BC, but its length is greater, being 477.9 inches. Its sectional area will therefore not differ much from that of BC. Using the value obtained for BC of $r = 8.00$ inches, $l/r = 60.0$, $p = 11\,840$ pounds per square inch, and the approximate sectional area is $613\,000/11\,840 = 51.78$ square inches. The following composition is adopted, as a test shows that no revision is needed:

1 cover plate, $26'' \times \frac{7}{16}''$	11.37 square inches.
4 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	11.48
2 web plates, $18'' \times \frac{9}{16}''$	20.24
2 flats, $5'' \times 1''$	10.00
Total	53.09

The wind stresses, according to the specifications, are not large enough to affect the area required to resist flexure in the plane of the truss, but must be considered in computing the stresses due to transverse flexure.

The end posts form a part of the portal which resists the wind pressure carried by the upper lateral system to the portal strut. The end posts bend in the plane containing their center lines. It is necessary then to compute the unit stress in the outer fiber of each end post due to its combined action as a column under the given direct compression and as a beam subject to the bending moment produced by the wind loads, and to compare it with the greatest allowable unit stress. If it exceeds the allowable value, the sectional area must be increased until the unit stress falls within the given limit.

The moment of inertia with reference to the axis CD in Fig. 120 is computed as follows, it being remembered that the backs of the angles are 19 inches apart:

$$
\begin{array}{lcr}
1 \text{ cover plate, } \frac{1}{12} \times \frac{7}{16}(26)^3 \quad \ldots\ldots\ldots\ldots & = & 640.8 \\
4 \text{ angles, } 4 \times 3.26 \quad \ldots\ldots\ldots\ldots & = & 13.0 \\
\qquad 11.48(9.5 + 1.04)^2 \quad \ldots\ldots\ldots\ldots & = & 1275.3 \\
2 \text{ web plates, } 2 \times \frac{1}{12} \times 18\left(\frac{9}{16}\right)^3 \quad \ldots\ldots\ldots\ldots & = & 0.5 \\
\qquad 20.24(9.5 - 0.28)^2 \quad \ldots\ldots\ldots\ldots & = & 1720.6 \\
2 \text{ flats, } 2 \times \frac{1}{12} \times 1 \times 5^3 \quad \ldots\ldots\ldots\ldots & = & 20.8 \\
\qquad 10.00(9.5 + 2.0)^2 \quad \ldots\ldots\ldots\ldots & = & \underline{1322.5} \\
\qquad\qquad I' \quad \ldots\ldots & = & 4993.5
\end{array}
$$

This computation indicates that the center line of each flat and the rivet line of the corresponding angle lie in the same vertical plane. Any eccentricity in the connection of the flats would develop secondary bending stresses. The radius of gyration with reference to the same axis is

$$
r = \sqrt{\frac{4993.5}{53.09}} = 9.70 \text{ inches.}
$$

Let the upper part of the end post be regarded as fixed at the lower portal strut by the portal bracing, and the lower end as fixed at the pin by the pedestal and the end floor beam. The drawing of the floor beam gives 4′ 8⅝″ as the distance from the base of rail to the bottom of the floor beam, and 10½″ from the bottom of floor beam up to the pin center. The clear head room required is 23 feet, and hence the distance from the center of the lower chord to the top of the clearance must not be less than 26′ 9⅞″. Allowing from 10 to 12 inches for the width of the lower portal strut, the inclined distance from the lower pin of an end post to the

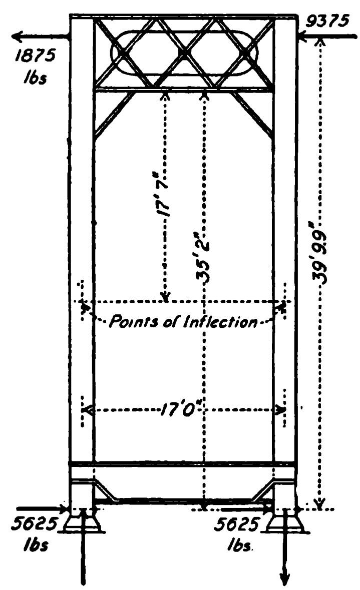


Fig. 121.

bottom of the portal strut is readily found by means of a diagram to be just 35 feet. Let the distance be taken as 422 inches. The point of inflection is at the middle of this length.

Fig. 121 indicates the action of forces on the portal, it being assumed that the reactions at the feet of the end posts are equal. The transverse bending moment in each end post is therefore $5625 \times 211 = 1\,186\,900$ pound-inches. By the first method given in Mechanics of Materials, Art. 102, the unit stress in the outer fiber is found to be

$$S_1 = \frac{Mc}{I - \frac{Pl^2}{6E}} = \frac{1\,186\,900 \times 14.0}{4993.5 - \frac{666\,700 \times \overline{422}^2}{6 \times 26\,000\,000}} = 3920 \text{ lbs. per sq. in.},$$

making the maximum compressive stress equal to $666\,700/53.09 + 3920 = 16\,480$ pounds per square inch. As this value does not exceed the specified limit of 19 000 pounds per square inch, no change is required in the composition of the end post.

The general formula given in the text-book just mentioned is the same as that quoted in Art. 90, except that the sign in the denominator is changed to minus. The form in which it is here given is adapted to the special case of the end post whose elastic line corresponds to that of one-half of a beam whose ends are fixed and loaded at the middle. Accordingly, $m = 192$, $n = 8$, and the l in the general formula equals $2\,l$ in the formula given in this paragraph.

A beam whose span is l, fixed at both ends, supporting a concentrated load Q at the middle, and subject to a longitudinal compression P, has a maximum compressive stress at the ends, or at the load, whose value is

$$S = \frac{P}{A} + \frac{Qc}{2\beta I} \tan \tfrac{1}{4}\beta l,$$

in which A is the area of cross-section of the beam, c the

distance from the neutral surface to the outer fiber whose unit stress is S, and $\beta = (P/EI)^{\frac{1}{2}}$, in which E is the coefficient of elasticity of the material. Applying this formula to the case of the end post, and remembering that l in this formula equals twice that used in the special approximate formula of the preceding paragraph, the true maximum compressive stress is found to be

$$S = \frac{666\,700}{53.09} + 3654 = 16\,210 \text{ pounds per square inch,}$$

or 270 pounds less than the approximate value first obtained.

ART. 94. LATERAL BRACING.

SPECIFICATION. — All lateral bracing shall be made of shapes which can resist compression as well as tension. In detailing struts composed of four angles with a single line of lacing, the clear distance between backs of angles shall never be made less than three-quarters of an inch, in order to permit the insertion of a small paint-brush. The stiff diagonals of the lower lateral system, of which there shall be two in each panel, shall be riveted rigidly to the stringers where they cross them, so as to transfer in an effective manner the thrust of braked trains to the truss posts without causing the floor beams to bend horizontally. In designing short members with riveted connections the sectional area of the piece shall be increased from 10 percent for 6″ × 3½″ angles to 25 percent for equal-legged angles beyond the theoretical requirements for the direct stresses, so as to compensate for the secondary stresses due to the eccentric grip of the rivets.

The most approved type of laterals for the upper system of through spans consists of two pairs of angles with one system of lacing between them, the depth of the member being equal to that of the upper chord. As the computed wind stress is only 13 300 pounds in one of the end laterals under the assumption that it resists the entire shear in the panel, it is clear that the stress alone cannot determine the section to be used. Since the principal duty of the lateral bracing is to resist the lateral vibration caused by the live load, and to hold the chord in line, it is important that they should have ample section to insure the

necessary stiffness. By using $\frac{3}{4}''$ rivets in the connections, and $\frac{5}{8}''$ rivets in the lacing, the angles may be reduced in size to $3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$, the longer legs being horizontal. For the required depth the lattice bars must be at least $\frac{3}{8}''$ thick, and hence the backs of the angles are spaced $\frac{3}{4}''$ apart. This makes the least radius of gyration 1.63 inches, and $l/r = 97.5$, the length from the center, where the laterals are riveted to a common connecting plate to the end connection, being about 159 inches. If the entire stress be resisted by one lateral as a column, the required area is 1.63 square inches, while only 0.89 square inch net section is needed for the same amount of tension. The section may then be adopted, although there is considerable excess of strength. If only two angles were laced together, they would have to be $4\frac{1}{2}'' \times 3''$ in size in order that the ratio l/r should not exceed 120, as specified for compression members. When the ratio falls beyond this limit, or when only single angles are used, they can be regarded merely as tension members, and the lateral system becomes correspondingly less efficient.

The net strength of the lateral when one rivet hole is deducted in each leg of the angles is $5.24 \times 15\,000 = 78\,600$ pounds, and hence the connections require 16 shop rivets or 20 field rivets $\frac{3}{4}''$ in diameter.

The stresses in the upper lateral struts are so small that their section is not determined theoretically. The student should consult some standard plans for bridges governed by approximately similar conditions and adopt the composition shown. See Plate IV for an example of a lateral strut which also forms the upper chord of the sway bracing.

Since the lower laterals are to be riveted to the lower flanges of the stringers, it is estimated that the greatest distance between the centers of connections is about 94 inches. In the first panel

the stress corresponding to the total shear is 80 000 pounds. Let the laterals be so designed as to resist either this entire amount in tension or half of this amount in both tension and compression, under the specification for alternate stresses, the wind stresses being treated as live-load stresses, but without any impact allowance. Placing two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$ back to back, the required area for compression is 4.29, while that for tension is 2.67 square inches, making a total of 6.96 square inches. The gross area furnished is 7.96 square inches, which covers an allowance of more than 12 percent for the effect of eccentric connections. Deducting one rivet from each leg of the angles for $\frac{7}{8}''$ rivets, the net section is 5.46 square inches, while a tension of 80 000 pounds requires only 5.34 square inches.

In a similar manner the laterals in the second panel require two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, while in the third and fourth panels two angles $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ may be used.

The connections require 13 shop or 16 field rivets in the first panel, 9 shop or 12 field rivets in the second panel, and 8 shop or 10 field rivets in the third or fourth panels, in order to develop the full strength of the net sections of the laterals.

The results of tests made by J. E. GREINER on the relative strength of single angles when connected by one or by both legs are recorded in Transactions American Society of Civil Engineers, vol. 38, page 63, Dec., 1897, while those made by C. F. LOWETH are given in Journal of the Association of Engineering Societies, vol. 8, page 268, May, 1889.

In order to comply with the specification which aims to prevent horizontal bending in the floor beams, due to the thrust of braked trains, WADDELL further specifies that the lateral diagonals and the stringers are to be made to form complete horizontal trusses by running angles between the stringers at the level

of the bottom flanges. Such angles are represented diagrammatically by the lines mm' and nn' in Fig. 122. As the equivalent uniform live load for a panel length of 25 feet is 9850 pounds per linear foot per track, the total thrust of the braked train per panel is $9850 \times 25 \times 0.20 = 49\,250$ pounds, the coefficient of friction being 20 percent (Art. 83). Dividing this between the two stringers, and assuming that the entire stress is taken by the truss $dnn'd'$, the stress in nn' is 16 800 pounds, and that in nd or in $n'd'$ is 26 800 pounds. If the truss $cmm'c'$ is assumed to act in conjunction with the other, the stresses will be divided accordingly. The smallest laterals have sufficient strength to take the stress of 26 800 pounds, either in tension or compression. A single angle, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, having a gross sectional area of 2.48 square inches, and a net area of 2.10 square inches, is found to have an excess of strength to provide for the eccentricity of its connections, even if it be subject to the entire stress of 16 800 pounds. The stress of 26 800 pounds requires 5 shop rivets or 6 field rivets, while that of 16 800 pounds requires 3 shop rivets or 4 field rivets. A detail of this kind and the connection of a lateral to one of the stringers may be seen on Plate VI.

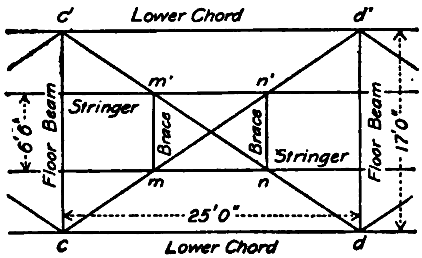


Fig. 122.

ART. 95. PORTAL AND SWAY BRACING.

SPECIFICATION. — All through spans shall have stiff portal bracing at each end, connected rigidly to the end posts. The bracing shall be made as deep as the specified clear head room will allow. When the height of the trusses is great enough to permit it, there shall be used at each panel point a rigid bracing frame riveted to the top lateral strut, and to the posts, and carried down to the clearance line. When the truss depth is not great enough for

this detail, corner brackets of proper size, strength, and rigidity are to be riveted between the posts and the upper lateral struts.

Fig. 123 gives a sketch of the general arrangement of the portal bracing to be designed, and the principal dimensions required to compute the bending moments to be resisted by the flanges of the lattice girder. The upper flange of the bracing

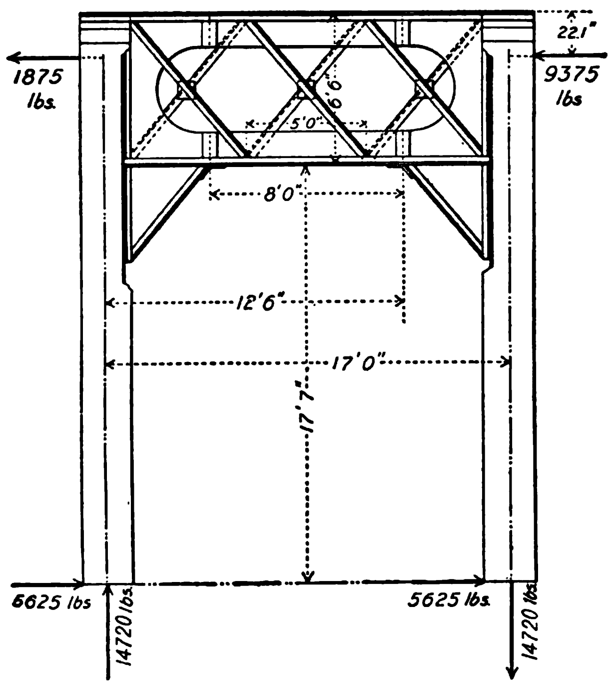


Fig. 123.

extends beyond the upper chords of the truss. The given forces are those obtained in designing the end post, and the reactions at the bottom of the figure are applied at the points of inflection of the end posts.

Let the shear be divided equally between the two diagonals cut by any section parallel to the end posts, and let the diagonals

be designed according to the specification for alternate stresses, the wind stresses being treated as live-load stresses for the same reasons as those given in the preceding article. The stress in each angle is then $7\,360 \times 1.2875 = \pm 9\,480$ pounds. Assuming one angle $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$, and the distance between centers of connections as about 38 inches, the ratio l/r is 61.3, and the sectional area for compression is found to be 0.81 square inch, while that for tension is 0.63 square inch, making the total less than that furnished by the angle, which is 2.30 square inches. It is not advisable, however, to reduce the size of the angle. The number of $\frac{7}{8}''$ rivets required in the connection of each web angle to the flange is $2 \times 9480/6610 = 3$.

According to the specifications the upper or compression flange of a plate girder is to be supported laterally at intervals not exceeding twelve times its width. The flanges of the portal bracing differ from those of a simple girder in having their stresses increase from the middle toward the ends, while those in the girder flanges increase from the ends toward the middle. The ratio of the laterally unsupported length of a flange of the portal bracing to its width may accordingly be taken considerably larger than twelve, say eighteen or twenty. This condition requires angles 5 inches wide. Assuming two flange angles $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, and neglecting the web plate on account of the splice at the section through the end of the bracket, the effective depth is found to be 76.1 inches.

With the forces shown in Fig. 123, the bending moments in a section against the inner web plate of the left-hand end post are $-1\,441\,000$ and $-1\,155\,000$ pound-inches, the centers of moments being in the neutral axes of the upper and lower flange angles respectively. When the forces are reversed in direction, the upper forces exchanging sides, the corresponding bending moments are $+1\,283\,000$ and $+1\,568\,000$ pound-inches. The area required to resist the alternate compression and ten-

sion in the lower flange is $1.26 + 1.12 = 2.38$ and in the upper flange $1.37 + 1.01 = 2.38$ square inches. The flanges must also resist a compression of 3800 pounds (Art. 87). The gross area of the assumed angles is 6.10, and the net area 4.60 square inches, one rivet hole being deducted from each leg of the angles. The angles, therefore, have more sectional area than necessary, but cannot be reduced in size without violating the specifications.

The statement made in the preceding article with reference to the design of the lateral struts applies to the entire intermediate sway bracing, of which the strut forms a part. The student should consult standard plans and make a comparative study of the details of the sway bracing. (See Plates IV and VII, and Art. 82.)

ART. 96. PIN PLATES.

SPECIFICATION. — Rivets shall not be countersunk in plates less than seven-sixteenths of an inch in thickness.

Pin plates shall be used at all pin holes in built members for the double purpose of reinforcing for the metal cut away and reducing the unit pressure on pin and bearing to or below the specified limit. They shall be of such size as to distribute properly, through the rivets, the pressure carried by such plates to both flanges and web of each segment of the member; and they shall extend at least six inches within the tie plates of said member, so as to provide for not less than two transverse rows of rivets there.

It is always better, whenever practicable, to avoid cutting away the ends of channels, but if they must be trimmed, the ends must be reënforced so that the strength of the member shall not be reduced by the trimming.

In riveted tension members, the net section through any pin hole shall have an area 40 percent in excess of the net sectional area of the body of the member. The net section outside of the pin hole along the center line of stress shall be at least 70 percent of the net section through the pin hole.

In designing the pin plates of the various members of the truss, it is necessary to observe not only the specification printed at the head of this article, but also the general one in Art. 83, which requires that in all main members having an excess of section above that called for by the greatest combination of

stresses, the entire detailing is to be done for the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected.

The maximum pin bearing at the bottom of the post *Cc* equals the maximum vertical shear in the diagonal *Bc*, and according to the rule just quoted, the value to be used in designing the pin plates of the post is the vertical component of the full working strength of *Bc*, which is $30 \times 15\,000/1.2847 = 350\,300$

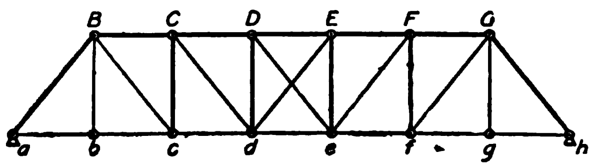


Fig. 124.

pounds, the sectional area of *Bc* being 30 square inches, and 1.2847 the secant of the angle which it makes with the vertical. As the diameter of the pin is 6 inches (Art. 91), the bearing required on each side of the post is $350\,300/(2 \times 6 \times 22\,000) = 1.327$ inches. The thickness of the channel web is 0.524 inch, and hence two pin plates are required whose thicknesses are respectively $\frac{7}{16}$ and $\frac{3}{8}$ of an inch. The outer pin plate cannot be less than $\frac{7}{16}$ inch, according to the specifications, since its rivets must be countersunk. If both plates be extended the same distance above the pin, the number of rivets required to connect them will be determined entirely by their bearing value in the channel web, or $0.875 \times 0.524 \times 22\,000 = 10\,090$ pounds for each rivet. The distribution of stresses between the pin plates and channel is in direct proportion to their respective bearings on the pin, and hence the stress taken by both pin plates is $0.813 \times 175\,100/1.327 = 106\,500$ pounds. Their full bearing value, however, is $0.813 \times 6 \times 22\,000 = 107\,300$ pounds, and therefore this stress is to be used according to the specifications. The number of rivets required is then

107 300/10 090 = 11. Fig. 125 shows the arrangement of the rivets. The outer pin plate on each side is extended to the foot of the post so as to act as a washer between the channel and the eye-bar *Bc*, and additional rivets are placed below the pin to keep the parts in contact.

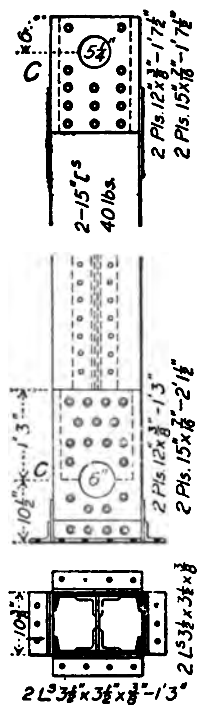


Fig. 125.

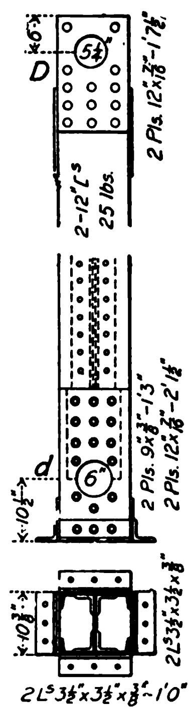


Fig. 126.

At the upper panel point the maximum bearing value on the pin is the full working strength of the post *Cc*, which is 23.52 × 11 110 = 261 300 pounds. This stress requires a bearing on each side of the post of 1.131 inches. Since the rivets in the outer pin plate must be countersunk, its thickness cannot be less than $\frac{7}{16}$ inch, and if this thickness be adopted, the inner plate must be $\frac{3}{8}$ inch thick, the minimum allowed. The full bearing value of both plates is 0.813 × 5¼ × 22 000 = 93 900 pounds, requiring 10 rivets to transmit their stress into the channel web. A symmetrical arrangement requires 11 rivets, as indicated in Fig. 125. The sizes of the pin plates and their riveting for the post *Dd* are given in Fig. 126.

Since the suspender *Bb* is a tension member, its net sectional area at the pin hole must be 40 percent in excess of the net area in its main body. The area for each side is therefore 13.80 × 1.40/2 = 9.66 square inches. The simplest arrangement is to use one pin plate 14″ × $\frac{9}{16}$″, giving a net area at the

pin of 10.24 square inches, but on computing the distance required beyond the pin hole it is found to be $6\frac{7}{8}$ inches, which exceeds the limit allowed by the upper chord. Using one pin plate $12'' \times \frac{7}{16}''$ on the outside, and another $9\frac{1}{2}'' \times \frac{3}{8}''$ on the inside, the net area obtained is 9.67 square inches. The net areas of the pin plates are 2.62 and 1.31 square inches, while their full tensile strengths are 39 300 and 19 650 pounds respectively. It is found that their bearing on the pin is below the specified limit. The value of a rivet in single shear is 6610 pounds, and its bearing in the web of the channel is $0.875 \times 0.513 \times 22\,000 = 9870$ pounds. If the inner plate be shorter than the outer one, it requires $19\,650/6610 = 3$ rivets. Their bearing value in the web is $9870 \times 3 = 29\,610$ pounds, leaving a balance of $19\,650 + 39\,300 - 29\,610 = 29\,340$ pounds to be taken by the additional rivets in the longer plate, and this requires $29\,340/6610 = 5$ rivets. If both plates have the same length, the number of rivets needed is $58\,950/9870 = 5$ rivets. To reduce the effect of eccentricity, both plates are lengthened somewhat beyond the limits indicated by the preceding computations. (See Fig. 127.) It must be remembered that in designing this member an allowance was made for two rivet holes in the flange and two in the web of each channel, and hence only two rivets are placed in any section below the top of the tie plates which are shown on the sides of the member. The distance beyond the pin is $9.67 \times 0.70/1.326 = 5\frac{1}{8}$ inches, according to the specifications.

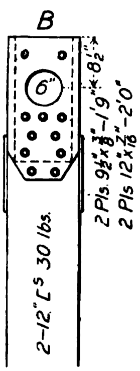


Fig. 127.

The net area at the pin holes in the diagonal *Cd* must not be less than $21.16 \times 1.40 = 29.62$ square inches, or 14.81 square inches for each side of the member. By turning outward the flanges of the channels the cutting needed to avoid interference

with other members of the truss is reduced to a minimum. At the lower end the flanges must be cut down flush next to the eye-bars, and hence two pin plates, each ½ inch thick, are required on each side. The net areas of the channel and of the inner and outer pin plates are 7.34, 4.50, and 3.00 square inches respectively, making a total of 14.84 square inches. Since the bearing of the rivets in the web of the channel, whose thickness is 0.72 inch, is greater than the double shear, the number of rivets in each pin plate is governed by single shear. The inner plate requires $4.50 \times 15\,000/6610 = 11$ rivets, and the outer one $3.00 \times 15\,000/6610 = 7$ rivets. But the full bearing value of the inner pin plate is only 66 000 pounds, which being less than its full tensile strength will slightly modify the

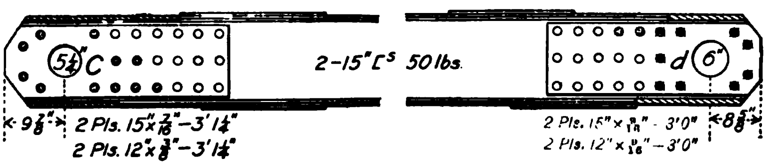


Fig. 128.

Fig. 129

distribution of stress beyond the pin. As several rivets are placed there to keep the plates in contact, they will have ample strength to transfer the stress. Since this member is also subject to compression, both pin plates will be made the same length in order to give additional stiffness to its forked ends. On account of this extension in length the number of rivet lines is reduced from four to three, and this requires a net area at the pin of 15.82 square inches, and a change in the thickness of the pin plates to $\frac{9}{16}$ inch in order to conform to the specification quoted from Art. 83. The required number of rivets is increased by one for each plate. The distance beyond the pin is $14.84 \times 0.70/1.97 = 5\frac{3}{8}$ inches. (See Fig. 129.)

At the upper end of *Cd* the flanges of the channels need only to be cut down to 2¼ inches, thus leaving a clearance of ¼ inch

between them and the upper chord with plates. The required sizes of the pin plates are marked on Fig. 128. The net areas of the channel, and of the inner and outer pin plates, are respectively 9.37, 4.27, and 2.53 square inches, making a total of 16.17 square inches. The stresses taken by the pin plates are 64 100 and 38 000 pounds, while the required numbers of rivets are 10 and 6 respectively. The plates are extended farther so as to pass the tie plates.

As shown in Fig. 130, the angles in the diagonal dE have to be cut off entirely one foot from the pin center in order to avoid interference with the post channels of Dd. The entire stress must, therefore, be carried to the pin by the pin plates. The full strength of dE is $17.88 \times 9210 = 164\,700$ pounds, since the area was determined by the compressive stress. The linear bearing on the pin for each side of the member cannot be less than $164\,700/(2 \times 6 \times 22\,000) = \frac{5}{8}$ inch. The full compressive strength of the member does not determine the net area required at the pin hole. If it be assumed that the entire shear in the panel causes tension in only one of the diagonals, the net area would be $152\,500/15\,000 = 10.17$ square inches.

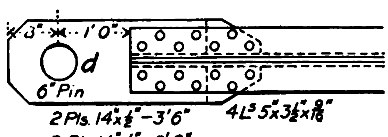


Fig. 130.

The corresponding area at the pin is, therefore, 7.12 square inches for each side, and if the pin plates be taken 14 inches wide, the total thickness must be 0.89, or say 1 inch, to allow something for excess of section in the member. Let two plates be used, each one-half an inch thick. The shorter one requires $41\,200/6610 = 7$ rivets, and both of them need $82\,400/6610 = 13$ rivets. The pin plates are to extend $7.12 \times 0.70/1 = 5$ inches beyond the pin. Let the same arrangement be used also at E. The pin plates are slotted and attached to the inner sides of the angles so as to reduce the effect of eccentricity.

The net sectional area of the stiff lower chord member which will be made continuous from *a* to *c* (Fig. 124) is 13 square inches on each side of the member. The composition of this section at the pin and the full tensile strength of each plate or shape are as follows:

	NET SECTION.	STRESSES.
1 web plate, 18″ × ½″	5.00 sq. ins.	75 000 lbs.
2 angles, 3½″ × 3½″ × ½″	5.50	82 500
1 pin plate, 11″ × ¼″	2.50	37 500
1 pin plate, 17″ × ⅜″	5.62	84 300
Total	18.62	

In determining the net section at the pin, two rivet holes are also deducted. (See Fig. 131.) At the end of the pin plates the web takes a stress of 112 500 pounds, and the two angles 82 500 pounds, three rivet holes being deducted from the web section and one from that of each angle. Since the web's share of the stress is just equal to that carried past the pin by the web as well as that of the narrower pin plate, the only stress that has to be transferred to the angles is that from the wider pin plate. The number of rivets connecting the latter to the angles must not be less than $84\,300/6610 = 13$, while only 6 rivets are needed to connect the narrower pin plate to the web. Although the angles extend past the pin, none of the rivets on the right of the pin should be counted in the 13 required. In order to avoid reducing the net section of the member, the rivets in the tie plates are given the same pitch as those in

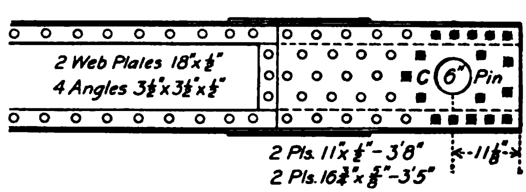


Fig 131.

the vertical legs of the angles until the pin plates are passed. On dividing the stress in proportion to the bearing on the pin, the $\frac{1}{2}''$ web and $\frac{1}{2}''$ pin plate take 60 000 pounds each, while the $\frac{5}{8}''$ pin plate takes 75 000 pounds. This stress due to bearing in the $\frac{1}{2}''$ pin plate exceeds that which it can carry past the pin by $60\,000 - 37\,500 = 22\,500$ pounds, and hence 3 rivets are required on the right of the pin to transfer this excess to the web and to the other pin plate. More than this number are inserted (Fig. 131).

The pin bearing at panel point *C* in the upper chord is to be designed to take the horizontal component of the full tensile strength of the diagonal *Cd*, or 199 200 pounds. The linear bearing on each side is $199\,200/(2 \times 5.25 \times 22\,000) = 0.863$ inch, and hence a pin plate of the minimum allowable thickness is required. As the web plate is $\frac{11}{16}$ of an inch thick, the pin plate's share of the bearing is $99\,600 \times 6/17 = 35\,200$ pounds. Since the only change in the section of the upper chord at *C* is in the web plates, the stress in the pin plate must be transferred to the web plate, and therefore requires $35\,200/6610 = 6$ rivets. Most of these are to be placed on the right-hand side of the pin, but in so small a plate the appearance is improved by making both sides alike, as shown in Fig. 132.

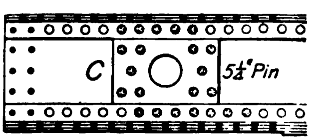


Fig. 132.

At the hip joint *B* (Fig. 124) the entire stress in the upper chord member *BC* and that in the end post *aB* are transferred to the pin, all the plates and shapes except the hinge or lap plates being faced parallel to the bisecting plane of the angle and about $\frac{1}{8}$ inch from it. The hinge plates of each member consist of two plates, located on the inside in one case and on the outside in the other, and extend past the pin. Their purpose is to prevent any accidental blow from displacing these

members, and to facilitate the erection of the truss. The combined pin plates on both members must be arranged with respect to each other so as to provide a clearance of at least $\frac{1}{8}$ inch between them. The full strength of the chord BC is $48.61 \times 13\,580 = 660\,100$ pounds, while that of the end post aB is $53.09 \times 11\,740 = 623\,300$ pounds. The linear bearings required for each side on a 6-inch pin are respectively $2\frac{1}{2}$ and $2\frac{3}{8}$ inches.

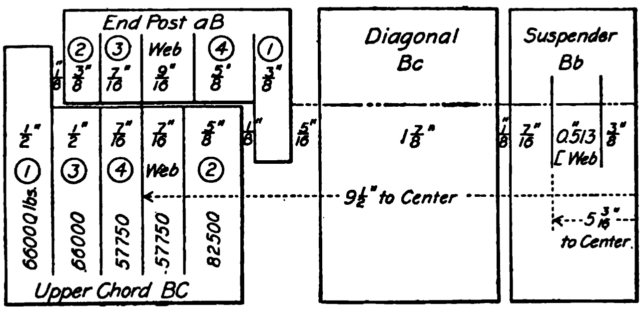


Fig. 133.

Fig. 133 shows the thicknesses and the arrangement of the pin plates, and the bearing stresses which they take. It is to be remembered that the distance back to back of the angles, or out to out of the web plates, is the same in both members. Beyond the pin plates the stresses in the plates and angles composing the chord BC must be directly proportional to their gross areas. Considering only one side of the member, the division of stresses is as follows:

	Gross Areas.		Stresses.
$\frac{1}{2}$ cover plate	5.685		
1 upper angle	2.87	8.555 sq. ins.	116 200 lbs.
1 web plate		7.88	107 000
1 lower angle	2.87		
1 flat	5.00	7.87	106 900
		24.305 sq. ins.	330 100 lbs.

Since nearly all the stresses in the pin plates must be transferred to the angles, the ideal arrangement of pin plates would be to have the same thicknesses on the outside of the vertical legs of the angles as (in symmetrical order) on the inside of the web plates, the plates outside of the angle being either of equal thickness or of regularly decreasing thickness. The plate next to the angle should be the longest and the outside one the shortest, those on the inside of the web being of the same successive lengths to make the entire arrangement symmetrical. Such a plan can be carried out completely in connection with the middle web of a chord having three webs, but with the outside webs it can only be approximated.

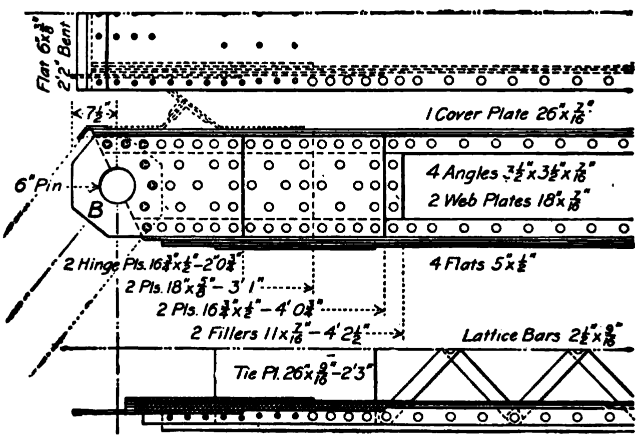


Fig. 134.

In this case the lengths are preferably made to alternate on the opposite sides of the web or angle, the filler plate being made a trifle longer than the next one on the outside merely for the sake of appearance. The object of this method is to transfer the stresses from the pin plates to the respective shapes composing the body of the chord by the most direct route and to put as many of the rivets in double shear as possible. Its application to the chord *BC* is illustrated in Fig. 134.

The web plate takes $107\,000-57\,750=49\,250$ pounds more stress than it gets directly from the pin bearing, and as the $\frac{7}{16}''$ plate, which also serves as a filler between the upper and lower angles, is not directly connected to the angles, it will be assumed that 49 250 pounds of its stress is transmitted directly into the web plate, and that the balance of its stress, or 8500 pounds, is to be transferred to the angles indirectly through all the other plates, including the web, in proportion to their respective thicknesses. The division gives plates 1, 2, and 3 (see Fig. 133) extra stresses of 2100, 2600, and 2100 pounds respectively. The total stress in plate 1 is then 68 100 pounds, and 11 rivets in single shear are required to transfer its stress to the upper and lower angles. Let the length of the plate be extended to include 6 rivets in the shorter angle, since its stress is to be divided about equally between the upper and lower angles. (See Fig. 134.) These 12 rivets also pass through plate 2, and being thus in double shear, their bearing in the angle will determine the stress which they can take out of both plates 1 and 2. This bearing value is $12\times8430=101\,200$ pounds, while the combined stress in both plates equals 153 200 pounds, leaving a balance of 52 000 pounds to be taken by additional rivets in single shear. The number required is $52\,000/6610=8$. Plate 2 is accordingly extended to engage 4 rivets in each angle beyond the extremity of plate 1.

The combined strength of plates 1, 2, and 3 is 221 300 pounds, while the bearing value of the 20 rivets which are in double shear is 168 600 pounds. The balance requires $52\,700/6610 = 8$ rivets in single shear. Plate 3 is therefore extended 4 rivet spaces beyond plate 2. The number of rivets required to carry the stresses from the filler plate 4 to the web and to plate 2 is $(49\,250+1800+2600)/6610=9$. Many more than this number must be inserted to keep the plates in contact and to give the necessary stiffness in compression. It will be noticed that

3 rivets are placed between the angles in the last vertical row in each pin plate, while in the other rows alternate rivets are omitted, the resulting pitch being the maximum allowed. The rivet pitch in the angles is 3 inches, and this pitch is to be continued far enough to satisfy the specification that at the ends of compression members the pitch shall not exceed four times the diameter of the rivets for a length equal to twice the width of the member. Beyond that limit the pitch is increased to a distance somewhat less than 6 inches, depending upon the lacing.

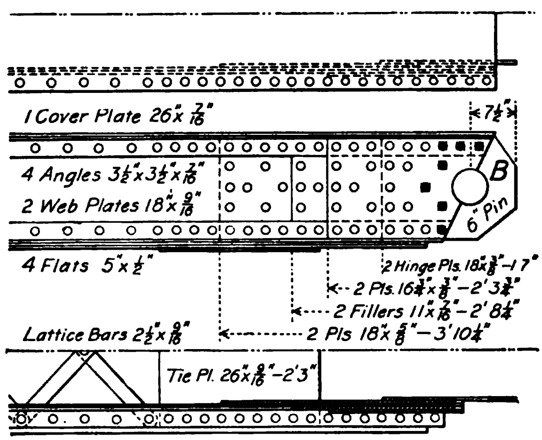


Fig. 135.

In the same manner the lengths of the pin plates on the end post and their riveting may be determined. Fig. 135 shows the result for the upper end of the end post. That for the lower end differs from this only in the effect of the additional vertical load transferred by the end floor beam (see Plate IV). In order to provide for the connection of the portal bracing, the middle row of rivets shown in Fig. 135 will be replaced by a double row of field rivets, the outer rows in the web plate being moved a little closer to the angles.

When a pin plate is shorter than its width, it is desirable to investigate it as a beam with its reactions at the rivet lines of the angles, and the load at the pin. In the case of the hinge plate on the end post it was found that a solid plate 13.6 inches long at its center is required. If the plate did not extend beyond the pin, its length would have to be increased.

In the upper chord and end posts of Fig. 111, Art. 82, the cover plate is 34 inches wide, and in order to avoid an excessive thickness of pin plates on the webs, short intermediate web

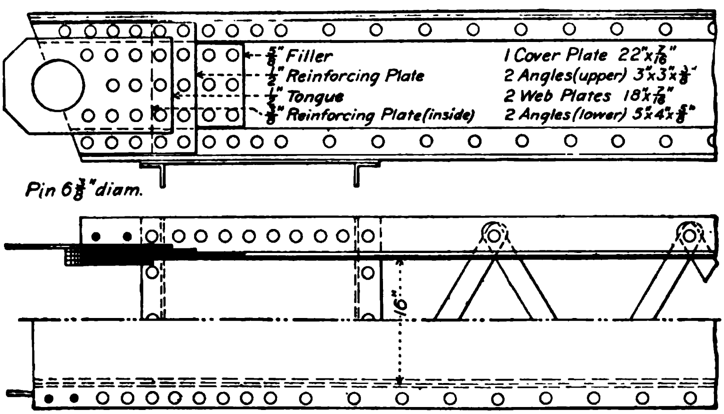


Fig. 136.

plates or diaphragms are inserted which also aid in a more direct distribution of stresses from the pin to the plates and shapes composing the member.

The results of some pin-plate tests are published in a paper by T. H. JOHNSON read before the Engineering Society of Western Pennsylvania, and reprinted from its Transactions in Engineering Record, vol. 28, page 39, June 17, 1893.

Fig. 136 shows an actual example which illustrates a frequent practice of inserting in the pin plates a sufficient number of rivets to carry their respective stresses out of the plates, but

without any regard to where those stresses are to be transmitted. The effect of this arrangement is to stress the web plates near the end of the member far beyond the safe value. It will be observed that only three rivets in the lower angles and four in the upper ones are in double shear, and that the wide $\frac{1}{2}$-inch outer pin plate should be considerably extended to engage additional rivets through the angles, while the inside plate should be made longer than any other one.

Another typical example of an inefficient design, but which gives an appearance of adequate strength, is that in which the filler plate is extended about twice as far as any other pin plate, and contains as many rivets as can be crowded in with a 3-inch pitch in both longitudinal and transverse directions.

ART. 97. TIE PLATES AND LACING.

SPECIFICATION. — At the ends of compression members the pitch of rivets shall not exceed four diameters of the rivet, for a distance equal to twice the greatest width of the member.

All segments of compression members connected by lacing only, shall have tie plates placed as near the ends as practicable. The tie plates shall have a length not less than the greatest width of the member, and a thickness not less than one-fortieth of the distance between the lines of connecting rivets, measured at right angles to the length of the member.

Single lattice bars shall have a thickness of not less than one-fortieth, and double bars connected by a rivet at the intersection of not less than one-sixtieth of the distance between the rivets connecting them to the members; and their width shall be:

For 15″ channels, or built sections with $3\frac{1}{2}''$ or 4″ angles	$2\frac{1}{2}$ inches ($\frac{7}{8}''$ rivets).
For 12″ and 10″ channels, or built sections with 3″ angles	$2\frac{1}{4}$ inches ($\frac{3}{4}''$ rivets).
For 9″ and 8″ channels, or built sections with $2\frac{1}{2}''$ angles	2 inches ($\frac{5}{8}''$ rivets).

The distance between connections of lattice bars shall not exceed eight times the least width of the segments connected.

In order to determine how close to the end of any member the tie plate — also called stay or batten plates — may be placed, it is necessary to draw the limiting outlines in direction as well as position of all the members which meet at the same panel point. Some clearance must be allowed so as to facilitate erection, and the riveting in the other leg of the angle or in the web of the channel, as the case may be, affects the location of the end rivet in the tie plate. The length of the tie plate must not only comply with the specification, but it is frequently made a little longer than the minimum limit so as to conform to the necessary spacing of the lattice bar connections. Where a tie plate is close to a web diaphragm of a member its length may be reduced. In tension members the tie plates are usually shorter than in compression members.

In members built up so as to require rivets between those connecting the lattice bars to the member, the space between adjacent connections is preferably a multiple of the rivet pitch, the latter not being expressed closer than a full eighth of an inch. In double lacing the multiple may be that of any number whether odd or even, but in single lacing the number should be an even one. In single lacing that on opposite sides of the member is arranged so that if both are projected on a parallel plane the combined projections are symmetrical about the central axis. The bars generally make an angle with a plane perpendicular to the axis of the member, not to exceed 30 degrees for single lacing nor 45 degrees for double lacing. Some specifications limit the distance between the connections to eight times the least width of the segments connected, or to the width of the channel plus nine inches. The spacing should also be such as to provide adequate openings for painting the interior surface of the member. In members of minor importance or in tension members the angles may slightly exceed these values. WADDELL'S specification mentions only single

lacing and prescribes lacing angles to be used when bars exceeding $2\frac{1}{2}'' \times \frac{1}{2}''$ would otherwise be required.

The American Bridge Company's Standards for Structural Details contains a table giving the maximum distances between connecting rivets for different thicknesses of bars, in accordance with the specification at the beginning of this article, the standard form and length of the ends beyond the rivets, and the ordered as well as the finished lengths of the bars. Examples of tie plates and lattice bars in both single and double lacing are shown in Figs. 111 and 134 and Plates IV, V, and VII.

Sometimes where the tie plates of posts cannot be placed very close to its ends, as in the case where the flanges are turned outward and cut off to clear the upper chord, short middle web diaphragms are inserted which extend to within a few inches of the pin. This diaphragm may be composed either of a channel or of a plate and two angles.

ART. 98. END BEARINGS.

SPECIFICATION.—Every span must be provided with some means of longitudinal expansion and contraction due to changes of temperature over a range of one hundred and fifty degrees Fahrenheit. Every span must be anchored at each end to the pier or abutment in such a manner as to prevent the slightest lateral motion, but so as not to interfere with the longitudinal motion of the trusses due to changes of temperature or loading.

The greatest allowable pressure upon expansion rollers of fixed spans, when impact is considered, shall be determined by the equation $p = 600\, d$, where p is the allowable pressure in pounds per linear inch of roller, and d is the diameter of the roller in inches. The least allowable diameter for expansion rollers is four inches. The bearings shall be so designed as to permit a free movement of the rollers in the longitudinal direction of the span sufficient to take up the extreme variations in length due to temperature changes and deflections, and at the same time prevent any transverse motion of the end of the span.

All shoe plates, bed plates, and roller plates are to be so stiffened that the extreme fiber stress under bending, when impact is included, shall not exceed 16 000 pounds per square inch. Bed plates shall be so proportioned that the

pressure upon masonry (including impact) will not exceed 400 pounds per square inch.

Pedestals shall be either of cast steel or built up of plates and shapes. In built pedestals, all bearing surfaces of the base plates and vertical bearing plates must be planed. The vertical plates must be secured to the base by angles having at least two rows of rivets in the vertical legs; and the said vertical plates must bear properly from end to end upon the base. No base plate, vertical plate, or connecting angle shall be less in thickness than three-quarters of an inch. The vertical plates shall be of sufficient height and must contain enough metal and rivets to distribute properly the loads over the bearings or rollers. The bases of all cast-steel pedestals shall be planed, so as to bear properly on the masonry or rollers. All rollers and the faces of base plates in contact therewith are to be planed smooth, so as to furnish perfect contact between rollers and plates throughout their entire length. All pedestals, whether built or cast, must have one or more diaphragms between webs, carried up as high as the general detailing will permit, so as to transmit any transverse horizontal thrust to the base without overstraining the webs by bending in their weakest direction.

The details of expansion bearing are described and illustrated in Arts. 44 and 81, while the design of such bearings was fully outlined in Art. 64 with reference to their use in plate girders. As the same principles apply equally to the design of the end bearings of trusses, the external forces, although larger, acting in the same manner, it seems unnecessary to extend the treatment of this subject. To explain and illustrate the design of all the details of the bearing of the truss under consideration in this chapter would also require more space than can well be spared for the purpose.

Attention should be called, however, to experimental investigations of the stresses in friction rollers. The papers named below give additional references to theoretical investigations and experiments. A Review of Professor Grashof's Investigation of the Carrying Capacity of Rollers and Balls, by CARL G. BARTH, may be found in Proceedings of Engineers' Club of Philadelphia for 1893, vol. 10, page 259. A valuable paper by C. L. CRANDALL and A. MARSTON, entitled Friction Rollers, is

published in Transactions of the American Society of Civil Engineers, vol. 32, page 99, Aug., 1894, with additional discussion on page 270. It contains the results of extended experiments on rollers of cast iron, wrought iron, and steel, and of the study of glass rollers under pressure by means of polarized light. A number of specifications have since been revised in conformity with the conclusion reached in this paper that the safe pressure varies directly as the diameter of the rollers, instead of as the square of the diameter, as implied in most of the specifications then in use. See also, MERRIMAN'S Mechanics of Materials, Art. 156.

The design of segmental rollers is discussed in the references given in Art. 81. See also a note on this topic by F. P. McKIBBEN, in Eng. News, vol. 36, pages 401 and 433, Dec. 17 and 31, 1896. The design of anchor bolts and cast pedestals is considered in an article by L. K. SHERMAN in Eng. News, vol. 55, page 137, Feb. 1, 1906.

ART. 99. MINOR DETAILS.

SPECIFICATION. — All plates, angles, and channels used in built members of trusses, must, if practicable, be ordered the full length of the member; otherwise the splices must develop the full strength of the member without any reliance being placed on the abutting ends for carrying compression. But in total splices at the ends of sections, perfect abutting of the dressed ends is to be relied upon. However, the splice plates even there must be of ample size and strength for both rigidity and continuity.

As shown in Fig. 111, and on Plates III and V, the upper chord is spliced a short distance to the left of a panel point in the left half of the truss. As the erection of the trusses begins usually with the middle panel, the chord is not spliced within the limits of that panel. In small trusses a splice is generally located in every panel except the middle one, but in larger trusses where the upper chord is horizontal it is often built in parts which are continuous over two panels. Such an arrange-

ment is shown on Plate IV, the web plates being spliced in the shop when the plates in the adjacent panels differ in thickness. The angles and cover plate are continuous, as their section is the same in both panels.

A splice plate is placed on both sides of each web, the outer one extending between the vertical legs of the upper and lower angles. Another plate is put on top of the cover plate, while the tie plate acts also as a splice plate below. All of these plates should be wide enough (longitudinally with respect to the chord) to permit two rows of rivets on each side of the joint. When the chord is large the size is increased, as shown on Plate V. In this case there are four webs, but as only the two outer ones can be spliced, one plate is placed on the inside and two on the outside of each of these two webs.

For the truss whose design is under consideration the splice plates on the side will be made 12 inches long, the joint being placed midway between consecutive rivets of 3-inch pitch in the vertical legs of the angles. The plate on the cover must therefore be 15 inches wide in order to have two rows of rivets on each side of the joint. The tie plate will have its ordinary length as stated in Art. 97. The elevation of one end of a chord member next to the splice was introduced for the sake of illustration in Fig. 132, Art. 96. The field rivets in the top and bottom beyond the first two in each line are for the plates connecting the laterals to the chord. As the chord is preferably built continuous from B to a point near D (Fig. 124), the web only being spliced in the shop, the chord joint shown in Fig. 132 really belongs to D, whose pin plates are made the same as those designed for C. Since the web plates are $\frac{7}{16}''$ thick in BC and $\frac{11}{16}''$ in CD, a filler $\frac{1}{4}''$ thick is required on one side of the web joint near C on the inside of each of the thinner web plates.

When the upper chord of a truss changes its direction in any two adjacent panels, the splice must necessarily be located directly at the pin, and the pin plates should then be designed to transmit the entire stress through the pin in the same manner as for the hip joint, no direct contact being allowed between the adjacent chord members. An important reference to this topic may be found in Proceedings of the Engineers' Club of Philadelphia, vol. 14, pages 155 and 164.

If possible, the stiff lower chord *ac* should have its sides parallel so as to simplify the construction by making all the lattice bars of equal length. It is found that the outside of its web plate is only three-sixteenths of an inch farther from the central plane of the truss than that of the end post. This distance must be increased three-quarters of an inch in order to permit the former web plate to pass the $\frac{3}{8}$-inch pin plate outside of the angles of the end post. At this end of the end post the outside pin plates will be made the hinge plates, so that the hinge plates of the pedestals will be on the inside. An inspection of the bending moment diagram of the pin at *c* shows that this change will slightly reduce the bending moment in the pin, provided the washer to fill the extra space on each side of the central plane is placed directly inside of the web plate, thus leaving unchanged the distances of all the eye-bars from the center except those of the two outer bars in the panel *cd*. Any other position of the washers would materially increase the bending moment and consequently change its diameter.

The diaphragm web between the channels of each intermediate post opposite the floor beam is designed to carry half of the floor beam reaction to the outer channel. The smallest angles allowed for $\frac{7}{8}$-inch rivets, namely, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles, and four in number, together with a web plate $9'' \times \frac{3}{8}''$, will furnish more than sufficient strength for this purpose. The length

of the diaphragm equals the total height of the angles connecting the floor beam to the post.

One of the diagonals of the middle panels must be cut so as to pass the other one, the two parts being connected by a plate on each side, the two plates having the necessary net section and number of rivets in single shear to transmit the full strength of the member. A short tie plate is inserted on each side of the splice. In a similar manner splices are to be designed for one of the upper laterals in each panel. The splice in the lower laterals consists of a single plate in each case. The computations required for these three sets of details are so simple that they will not be given.

ART. 100. CAMBER.

SPECIFICATION. — All trusses must be provided with such a camber that with the heaviest live load on the span, the total camber shall never be quite taken out by deflection. With parallel chords sufficient camber will be obtained by making the panel of the top chord longer than those of the bottom chord by one-eighth of an inch for each ten feet of length. The increased length of the top chord shall be neglected in figuring the lengths of main ties, but shall be considered when figuring the lengths of inclined end posts and counter-braces. Half the increase in length shall be considered in figuring the length of top laterals. One-half of the camber after a span is swung is to be taken out of the track by notching the ties, unless this would cut too deeply into the timber.

The application of this specification to the truss under consideration makes the actual or shop lengths of the upper chord panels $\frac{5}{16}$ inch longer than the nominal length of 25 feet. The length of the main diagonal, without allowance for the clearance of the pins in the pin holes, is 39 feet $10\frac{1}{32}$ inches. The length of the end post and counter-braces is $\frac{1}{16}$ inch longer without a similar allowance. To the several lengths just given $\frac{1}{32}$ inch is to be added for pin clearance in the case of compression members, while it is to be subtracted in the case of tension members.

A more precise method of providing for camber, and which must be applied to larger spans according to the above specification, consists in making the shop length of each tension member shorter than its nominal length by an amount equal to the elongation caused by its stress under the dead and full load when increased by a small percentage. The length of a compression member is correspondingly increased. The same allowance for the pin clearance is to be made as that noted in the preceding paragraph. When the elongation or shortening is computed for the dead and full live load stresses only, the camber is entirely taken out under the full live load. The lengths of secondary members, like the short diagonals in a Baltimore truss, are sometimes made the mean of the nominal length and that obtained in the manner just described.

The deflections at the various panel points are most conveniently found by the graphic method explained in Chap. VII, Part II. The results may be checked either by a separate diagram or by computing the deflection at the middle panel point of the loaded chord, by the method given in Chap. VII, Part I.

In an article entitled Camber of Bridges, in Railroad Gazette, vol. 22, page 665, Sept. 26, 1890, THEODORE COOPER states the object of camber and describes its relation to the track surface. See also Camber in Bridge Trusses, by G. H. PEGRAM, in Engineering News, vol. 18, page 21, July 9, 1887.

Special attention was given in the design of the Delaware river bridge to its deflection and camber. The lengths of the members were so arranged that the center of the bottom chord should be about as far below its normal position under a full load on both tracks as when the span is unloaded, it being assumed that the greatest live load on the bridge would very rarely exceed that of a full load on one track. A description of the methods employed to secure this result, to avoid excessive

tension in the stringer connections due to the elongation of the lower chord, and to reduce the effect of secondary stresses due to the subdivision of the panels by the secondary web system, is given by PAUL L. WÖLFEL, in Proceedings of the Engineers' Club of Philadelphia, vol. 14 (1897), page 156.

ART. 101. ANALYSIS OF WEIGHT.

SPECIFICATION. — If in any bridge design the dead load assumed shall differ from that computed from the diagram of sections and the detail drawings by an amount exceeding one percent of the sum of the equivalent live load and actual dead load, the calculations of stresses, etc., are to be made over with a new assumed dead load.

After computing the weight of every member the results for one truss, exclusive of the pedestals, may be classified as follows:

TRUSS MEMBERS.	POUNDS.	ONE-HALF LATERAL AND TRANSVERSE BRACING.	POUNDS.
Intermediate posts	11 632	Upper laterals and	
Suspenders	5 406	connections	5 922
Diagonals	26 162	Lateral struts and	
Lower chord	29 924	sway bracing	2 686
Upper chord	28 682	Portal bracing	3 604
End posts	18 134	Lower laterals and	
Pins	2 632	connections	7 410
	122 572		19 622

If the pins be included with the chords, the weight of the web members is found to be 61 334 pounds, and that of the chords to be 61 238 pounds, thus indicating that the depth chosen is the one which makes the weight of the truss a minimum.

The total weight of these members is made up of the following items:

	POUNDS.	PERCENT.
Main shapes and plates composing members	109 768	77.2
Pin plates	7 608	5.3
Tie plates and lacing	10 676	7.5
Connections, splices, and other details	10 460	7.4
Rivet heads	3 682	2.6
	142 194	100.0

In the first edition of Part III a corresponding analysis was given for a single-track Pratt truss bridge with a span of 142 feet, and designed for a load but little more than half that specified in Art. 83. The corresponding percentages were 76.7, 5.3, 7.1, 6.1, and 4.8. This shows that the relative combined weight of the details is only slightly affected by considerable changes in the loading and specifications.

Since the net weights of the shapes and plates composing some of the members cannot be computed with precision until many of the details are designed, it is desirable to compare the total weights of the several classes of members with their theoretic weights obtained by means of the adopted gross sectional areas and their lengths, center to center of pins, no deduction being made for pin holes. Such a comparison is made in the following table:

TRUSS MEMBERS.	FINAL WEIGHTS.	THEORETIC WEIGHTS.	RATIO.
Intermediate posts and suspender	17 038	11 776	1.447
Diagonals and stiff chord *ac*	33 028	23 352	1.414
Upper chord and end posts	46 816	37 334	1.254
Eye-bars	23 058	20 026	1.151
	119 940	92 488	1.297

These ratios vary somewhat for various types of pin trusses and for different spans, but the difference is comparatively

small. For instance, in one of the fixed spans of the Delaware river bridge, whose length is 533 feet, the upper chord being curved and the panels subdivided (see Fig. 10), the ratios are as follows: Intermediate posts and long suspenders, 1.457; upper chord and end posts, 1.204; eye-bars, 1.137; sub-verticals, 1.975; intermediate horizontal stays or rails to support the long posts, 1.839; and total, 1.228. These values are given by F. C. KUNZ in the article to which reference is made in Art. 82.

By means of such ratios the dead load assumed in computing the stresses may be corrected as soon as the sections of the members are designed, thereby avoiding any revision after the details are designed. No revision was made in this chapter in accordance with this method in order to furnish the data for an example to the student, who should make the revision and observe the consequent effect upon the sections of the members.

The dead load for one span, exclusive of the pedestals, is divided as follows:

	POUNDS.	PERCENT.
Track	77 000	16.4
Steel floor system	107 240	22.9
Trusses and connecting bracing	284 390	60.7
	468 630	100.0

The steel floor system is made up of two end floor beams, each weighing 3537 pounds, four brackets outside of the end floor beams and in line with the stringers, each weighing 303 pounds, together with fourteen stringers and six intermediate floor beams whose weights are given in Arts. 85 and 86.

The excess of the actual dead panel load per truss over that assumed is 33 470 − 30 000 = 3470 pounds. The sum of the equivalent live load and the actual dead load per panel is 79 100 + 33 470 = 112 570 pounds. The excess of the dead panel load is 3.08 percent of this sum, and hence a revision is required according to the specification printed at the head of this article. It will be found that only a part of the sections of members need to be increased on account of their excess of area over that previously required.

ART. 102. GENERAL DRAWING.

Instead of reproducing the general drawing of the truss and its connecting bracing, whose design is given in this chapter, there is inserted in Art. 82 a similar drawing of a single-track through truss bridge of a span of 160 feet, it being a part of one of the standard plans of the Northern Pacific Railway prepared by RALPH MODJESKI (see Plate IV). Four other sheets, not reproduced, belong to the complete set.

The dead load is assumed to be 2550 pounds per linear foot of bridge, one-third of it being concentrated at the upper panel points. The live load consists of two 188.5-ton locomotives with a combined length of 114 feet, followed by a uniform load of 5000 pounds per linear foot of track. The top lateral bracing is designed for a wind load of 200 pounds per linear foot, and the bottom lateral bracing for a static load of 250 pounds and a moving load of 300 pounds per linear foot.

The impact specified is practically 100 percent since the unit-stress for the live load is one-half that for the dead load. It is noted, however, that the dead load unit-stress is higher than that in the specifications of the American Railway Engineering Association. For web members in compression and for all tension members except hangers, the live load, however, is increased by

an excess load percentage which equals the ratio of the unloaded length to the span. For hangers and members subject to sudden loading, as well as for compression members receiving direct stress, the live load is increased by an unbalanced load percentage which equals $\frac{1}{8}(100 - l)$ in which l is the span in feet. The same addition is made for the flanges and webs of floor beams and stringers. In members subject to reversal of stress the rivets are proportioned for the sum of both kinds of stress, while the pin bearings are increased from 60 to 100 percent over what is required for the maximum stress.

The least transverse dimension of the upper chord and end post is not to be less than one-sixteenth of their respective insupported lengths, and that portion of the top cover plate which is unbalanced by the bottom flanges is not to be included in determining the sections of these members.

The stringer (not shown on Plate IV except in section) has a web plate $42\frac{1}{2}'' \times \frac{1}{2}''$ without any intermediate stiffeners, and flanges composed of two angles $6'' \times 6'' \times \frac{3}{4}''$. The connections at each end consist of two angles $6'' \times 6'' \times \frac{9}{16}''$ and two fillers $9'' \times \frac{3}{4}''$. The laterals are composed of single angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, arranged like the web members of a Warren truss, in a continuous series from end to end of the span, there being three panels of the lateral bracing in each panel of the bridge. The bracket in each line of stringers outside of the end floor beam projects 15 inches beyond the center of the floor beam.

All the material in the span, except where otherwise noted, is medium steel. All rivets and bolts are of soft steel, the former being $\frac{7}{8}$ inch in diameter. The first dimension in the size of any angle which is marked on the drawing indicates the width of the leg shown.

The student's attention is called to the following features which differ from those indicated in Figs. 113 to 135 inclusive: the

construction of the end of the intermediate floor beam and its clearance for the eye-bars; the wide spacing of the stringers; the addition of a collision strut which in turn necessitates a pin connection at panel point L_1; the position of the eye-bars at U_1 on the outside of the upper chord, thus reducing the width of the chord; the absence of a cover plate in the upper chord; the uniformity in size of all the pins except that at L_1; the composition and arrangement of the upper lateral diagonals. Some other items were referred to in Chap. VIII.

The student should carefully study the details of other modern designs from the blue prints in the college collection, or by visits of inspection to actual bridges in the vicinity, and record in his note book the special features of the construction in each case. If this is done in some regular order, many points will be noticed that otherwise would be overlooked. The study of shop drawings on which each member is shown separately in the manner described in Art. 17 is especially important with reference to the location of rivets, their relation to center lines and to points of intersection of axes of connecting members, and their influence on the exact lengths of the projecting ends of members. They also show modifications in spacing to avoid interference, and what rivets are flattened or countersunk to secure the necessary clearance. These are sometimes not shown on general drawings.

The remaining sheets included in the set of standard plans with Plate IV, which is designated as R-10-1076, consist of No. 1075, the stress sheet; No. 1077, which shows the details of the stringers and their lateral bracing, the stringer bracket, and the turned bolts required at the top of the stringer connection to the floor beams, as well as the general relations of the expansion end of the truss to the masonry, of a double fixed end, and of a combined expansion and fixed end; No. 1078, which gives

the details of the fixed and expansion bearings; and No. 276, which shows standard hook, floor, and anchor bolts.

ART. 103. BRIDGE DESIGN REFERENCES.

In Art. 7 references are given to the principal literature on bridge design. The following articles which have appeared in the engineering periodicals will also repay careful reading.

Bridge Design. By H. J. LEWIS. Eng. News, v. 26, p. 367, Oct. 17, 1891. Bridge Details. By E. SWENSSON. R. R. Gaz., v. 24, p. 156, Feb. 26, 1892.

Advance in the Design of Bridge Superstructure. By G. S. MORISON. Eng. News, v. 30, p. 80, July 27, 1893.

Details of Construction of Engineering Structures. By C. C. SCHNEIDER. Eng. Rec., v. 32, pp. 256, 364, 382, Sept. 7, Oct. 19, 26, 1895.

Some Hints on Bridge Designing. By OSCAR SANNE. Jour. W. Soc. Engrs., v. 4, p. 229, Apr. 1899.

Excessive Refinement in Bridge Design. Editorial. Eng. Rec., v. 44, p. 393, Oct. 26, 1901.

Bridges for Electric Railways. By C. C. SCHNEIDER. Street Ry. Jour., v. 28, pp. 398 and 441, Sept. 15 and 22, 1906.

Some Commercial Features of Structural Engineering. By EMIL GERBER. Proc. Engrs. Soc. W. Pa., v. 23, p. 125, Apr., 1907.

Proportioning of Steel Railway Bridge Members. By H. S. Prichard. Proc. Engrs. Soc. W. Pa., v. 23, p. 324, July, 1907. Discussion on p. 573, Dec., 1907.

CHAPTER X.

HIGHWAY BRIDGES.*

ART. 104. CLASSES OF BRIDGES.

Highway bridges are classed according to the character and weight of the loads which they are required to carry, but this classification is for the greater part arbitrary. Three general classes are: (1) bridges for city traffic; (2) bridges for city and electric car traffic; (3) bridges for country traffic. These are in turn divided into other classes according to the weight of the traffic which crosses them.

Bridges for city traffic are, in addition to the weight of the people who may crowd upon them, liable to have heavy loads pass over them, such as street rollers or trucks carrying heavy structural members or machinery, and all parts of the bridge and its floor must be designed accordingly. If electric cars pass over them, parts of the floor system, and likewise the trusses, must be of sufficient strength in order to sustain this traffic in addition to the other loads which may be upon the bridge at the same time.

Country bridges must be able to carry the heaviest load of farm produce or the heaviest piece of farm machinery, in addition to persons who may pass over them. It is customary to assume the people to act as a uniform load, and to take for the roller or truck a certain system of loads which will give stresses equal to any loading which may pass over the bridge.

* By F. O. DUFOUR, C.E., Assistant Professor of Structural Engineering, University of Illinois.

The uniform load which represents a crowd of people is in some states prescribed by law, otherwise it is determined by the engineer or taken from any of the so-called standard specifications for highway bridges, of which those by THEODORE COOPER and J. A. L. WADDELL are among the best. The engineer should not allow himself to adhere too closely to these 'standard' specifications in respect to the loading. He should carefully consider the locality and be particular to ascertain the heaviest loads which pass over the bridges in the vicinity or are liable to do so in years to come. He should then choose a 'class' from those given in the specifications, or when necessary, in some cases, take such a loading as the case demands even though it differs from those which are given.

ART. 105. SPECIFICATIONS.

The engineer may write his own specifications, if he is experienced enough or has a bridge of exceptional importance. In the great majority of cases he will do well to adopt one of those written by experienced engineers and published in convenient pamphlet form.

Some of these specifications specify an allowance for impact. According to experiments made on existing highway bridges it seems inadvisable, no matter how short the span, to use impact allowances unless the bridge has a plank floor. Reinforced concrete floors are of sufficient weight to prevent impact except in the case of light floors; and in such cases the sum of the static and impact unit-stresses does not approach the allowable unit-stresses which are used in the design.

In general, most specifications are incorrect in their loading for the floor system of the class which represents country bridges. A load of from 6 to 12 tons is usually specified. This is supposed to represent the farm or traction engine used for

many purposes. In reality some farm engines weigh as much as 30 tons; such are exceedingly rare, however, and the assumption of a 15-ton engine is on the side of safety in most cases.

The uniform load which represents a crowd of people is, in most specifications, less for the longer than the shorter spans. This is as it should be, since a short-span bridge is liable to be more densely crowded than one of long span. In such cases care should be taken to design a 'legal' bridge, that is, one which confirms to the state law. In some states the law requires the uniform load used in designing to be 100 pounds per square foot of floor surface, irrespective of the length of the span.

Clauses requiring the first two panels of the lower chord in pin-connected trusses to be built-up members should be used only in case the bridge is to have a plank floor. If a concrete floor is to be used, this in itself will give the requisite stiffness to the structure.

ART. 106. FLOOR SYSTEMS.

Previous to 1900 few bridge floors were made of other material than wood. The country highway bridges had floors of plank which rested directly on the stringers, being fastened

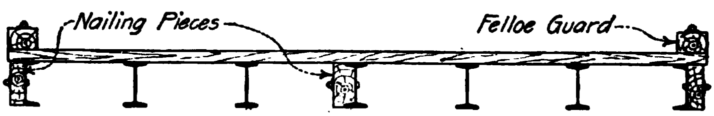


Fig. 137. — Wooden Floor.

down by nails to nailing pieces which ran longitudinally and were usually bolted only to the outer and center stringers, as shown in Fig. 137. These nailing pieces were generally 4 inches wide and as deep as the joist.

In some cases a felloe guard was used, one being on each side. The use of these felloe guards has been discontinued

because they served as a lodging place for dirt, while rain caused continued dampness between them and the floor, which soon rotted the floor plank at the ends and so caused them to become useless. A renewal then became necessary although the planks were perfectly good except at the ends. In some cases these felloe guards were raised one inch from the floor by means of washers at the places where they were bolted down. This was supposed to prevent rotting, but the dirt filled up the space and decay frequently took place sooner than in cases where the felloe guards were bolted down tight. The floor plank usually had a thickness in inches equal to the spacing of the joists in feet, and were laid so as to have one-half inch spaces between them. This space was supposed to allow for drainage and to permit a large portion of the dirt which was brought upon the bridge to fall through.

In some cases a floor was laid as described above, and another one, called a wearing floor, was put on top of it. The plank was laid diagonally on one of the floors and perpendicular to the axis of the span on the other.

Several firms manufacture a patented wooden floor. While these patented floors are superior to the usual plank floor (Fig. 137) and last considerably longer, it is doubtful whether their use is advisable. This question should be settled for each case. It is suggested that nothing is to be gained by their use in place of reinforced concrete until the span is of sufficient length so that the weight of concrete requires additional metal in the bridge, the cost of which capitalized will more than pay for the renewals of the patented floors at intervals.

All wooden floors have a common disadvantage in being difficult to attach to joists so that they will remain secure. They are liable to become loose under traffic, and to cause accidents to horses on account of catching their feet.

In case the bridge is in a city, the floor should be of the same construction as the paving of the adjacent street. Bridges with floors of brick, Belgian block, asphalt, or of various patented forms of construction have been built, and are giving satisfactory service. Fig. 138 shows a brick floor which is in use on a city bridge, and is giving good satisfaction. It is typical of the construction of the kinds of floors mentioned above. In all

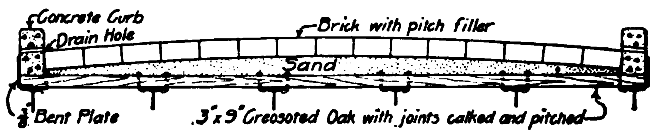


Fig. 138. — Brick Floor.

cases, a timber, reinforced-concrete, or iron covering is placed over the joists, and on top of that a cushion of sand. The wearing surface is placed on the sand cushion.

Great care should be taken to allow for longitudinal and transverse expansion. Wide joints between the bricks will usually be enough to allow for transverse expansion. This is not, however, sufficient for longitudinal expansion. To allow for this, joints at every 25 feet of span and running completely across the bridge should be used. They should be at least one inch wide and filled with an asphalt or a pitch matrix.

The majority of floors of country highway bridges are now built of concrete or reinforced concrete with an earth cushion on top. Plain concrete floors are as a rule uneconomical on account of the fact that their thickness must be great and the spacing of the joists small in order that the concrete may be safe in tension. Figure 139 shows two forms of concrete floors. The thickness of the first one is determined by considering it as a simple beam. The second floor is designed by considering it as a no-hinged arch. To keep the joists from spreading, tie

rods should be placed at the centers of the webs and spaced about 10 feet apart. Figure 140 shows the details of two methods, that in Fig. 140*a* being the most economical. The use of plain concrete floors is not recommended.

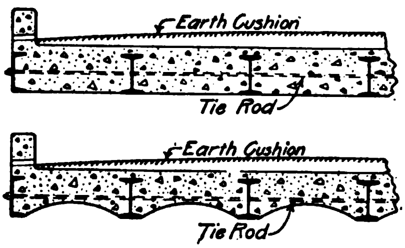


Fig. 139. — Plain Concrete Floors.

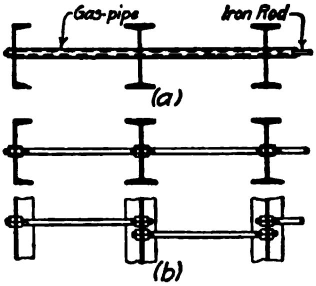


Fig. 140. — Details of Joist Tie Rods.

Floors of metal troughs and concrete are used to some extent; in most cases, however, they serve as a base for a wearing surface of brick, cobblestone, or Belgian block. In these floors some type of the usual metal solid-floor sections, buckle plates, or one of the many patented sections are used in connection with longitudinal joists. The solid-floor sections, shown in Fig. 141, are designed according to their moments of inertia. The details of part of a cross-section are shown in the Fig. 141*a*. A 3-inch crown, at least, should be given to the concrete, while sufficient provision for drainage is made. In case it is inadvisable to allow

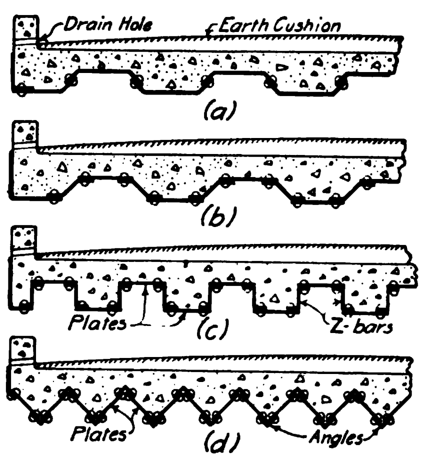


Fig. 141. — Solid Floor Metal Sections.

the drippings from the drainage pipes to fall freely beneath the structure, they may be led to desirable places by means of down spouts. At least one drainage hole on each side of the roadway should be used in every panel.

Floors of buckle plates and plain concrete are shown in Fig. 142. The use of this, as well as other types of solid-floor sections, is not recommended except in cases where the loads are so excessively heavy as to require reinforced concrete floors of great weight. Their cost is greater than that of reinforced concrete floors for ordinary traffic.

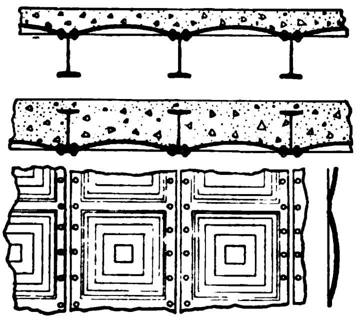


Fig. 142. — Buckle Plate Floors.

Many patented metal floor sections are on the market. The Buckeye trough and the Multiplex, shown in Fig. 143, are examples of this class of floor section. Their use permits the most economical spacing of the joists for any given loading, since these floorings may be cut to

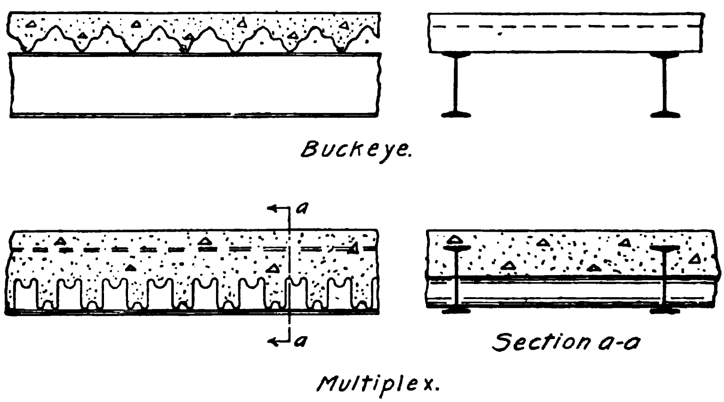


Fig. 143. — Patented Metal Floor Sections.

almost any desirable length. The trade catalogues of the companies manufacturing these sections give the spacing of joists for different loadings.

Reinforced concrete floors for highway bridges have come into general use since 1905. The price of oak of the requisite quality for bridge floors has increased considerably since 1895 and is still increasing. The cost of cement has decreased considerably in the same time. As matters stand in 1911, the reinforced concrete floor is by far the most economical in the prairie states and in all others which are thickly populated. Of course, in some regions where timber is cheap and plentiful or the haulage for cement is long, timber may be the most economical even if frequent renewals are necessary. The cost of any one of the several reinforced concrete floors described in this article will not vary greatly in first cost from an oak floor 3 inches thick. In twenty years the cost of a reinforced concrete floor will double with compound interest at 3½ percent, while that of a plank floor will be five times the first cost on account of renewals and interest charges. A reinforced concrete floor, on account of its additional weight, will require additional material in the floor system and in the trusses. This will tend to decrease the favorable showing of the reinforced concrete floor just mentioned, but it has been computed by the author of this chapter that at the end of 15 years it will be ahead of the plank floor, all conditions considered.

The floor shown in Fig. 144 was designed by IRA O. BAKER, Professor of Civil Engineering in the University of Illinois. For economy in joists their maximum spacing should be about 3 feet. To determine the sizes of joists the bending moment caused by the roller is assumed to be resisted by four joists spaced 2 feet between centers. If the joists are spaced more than 2 feet, the moment is assumed to be taken by three joists.

This floor has been used on bridges in Illinois and adjoining states since about 1900 and has given universal satisfaction.

State Engineer A. N. JOHNSON of Illinois uses the floor shown in Fig. 145, which is designed for a 15-ton engine. Since this

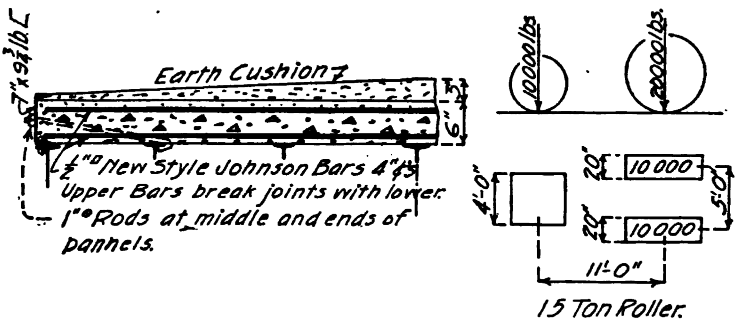


Fig. 144. — Baker Floor and Loading.

floor is reinforced in both directions, the assumption made to determine the size of the joists is that the bending moment caused by the engine is distributed over all the joists. The maximum spacing of the joists is about 3 feet.

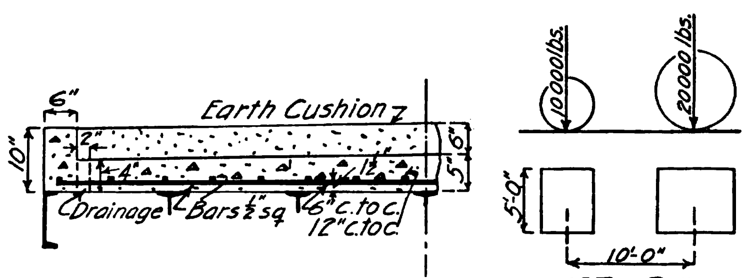


Fig. 145. — Johnson Floor and Loading.

J. E. KIRKHAM, Bridge Engineer for the State Highway Commission of Iowa, uses the floor and loading illustrated in Fig. 146. The maximum spacing for joists is the same as in the other floors just mentioned, or 3 feet. KIRKHAM considers

the entire weight of the engine as a uniform load distributed over a space 12 feet long by 7 feet wide. As a matter of fact, experiments performed by the author with a wheel load of 6600 pounds, the tread of the wheel being 10 inches, indicate that the

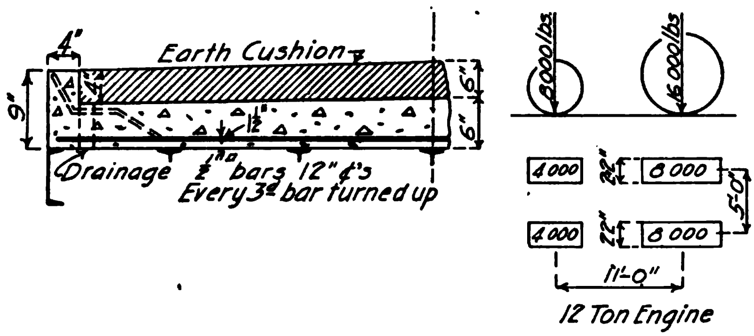


Fig. 146. — Kirkham Floor and Loading.

joist developing the greatest stress supports 40 percent of the load in plank floors and 25 percent in reinforced concrete floors.

If the panel length does not exceed 10 feet, the steel joists may be omitted and a reinforced concrete slab used instead. For a 16-foot clear roadway, this will require 18-inch, 50-pound I-beams for floor beams. Figure 147 shows a floor of this char-

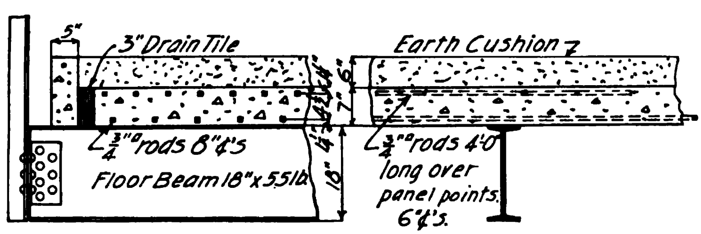


Fig. 147. — Reinforced Concrete Floor.

acter. It was also designed by KIRKHAM, has been used by him and the writer, and found to be very economical and satisfactory. It is adapted to the use of pony Warren trusses with

subverticals, thus keeping the panel length within the required limit. A 12-ton engine is used. In connection with this floor, stiff lateral bracing should be employed. Single angles, $2\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$ inches, will usually be sufficient.

In some states of the West, as well as in other localities, there are many drainage ditches. These ditches require occasional dredging, and hence the bridges must be taken down to let the dredge pass through. In such cases the solid reinforced concrete floor is a disadvantage. To meet this difficulty a removable reinforced concrete floor is frequently used. Blocks, reinforced in both directions on top and bottom to prevent breakage from handling as well as from the traffic, are made of the required thickness in lengths that can be handled without special machinery. These blocks rest directly on top of the joists (see Fig. 148), while the earth cushion is placed directly upon them. Allowance must be made for a $\frac{3}{4}$-inch spacing between the blocks or otherwise they may not fit, on account of having spread the forms during construction. A looped rod is placed in the center of each section so that it can be handled easily. When the time comes to clean out the drainage ditch, these slabs are lifted off, the bridge is taken out, the dredge passes through, the bridge is then reërected and the slabs put back in place.

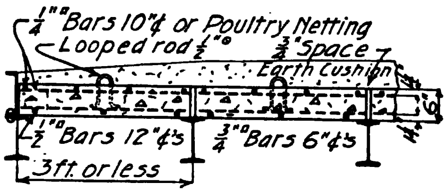


Fig. 148. — Removable Reinforced Concrete Floor.

In designing any of the reinforced concrete floors mentioned above, their dead load, including reinforcement and earth cushion, may be assumed as 125 pounds per square foot. This takes into account the fact that when the earth cushion becomes

saturated with water it is impossible to have heavy engines pass over it, since they would slip about too much to allow their successful operation.

ART. 107. BEAM BRIDGES.

Beam bridges are constructed by placing beams side by side and spaced from two to three feet apart. Both ends of these beams rest on the abutments. The outer beams usually consist of channels while the rest are I-beams. The floor is either of plank laid crosswise on the beams or one of the floors described in Art. 106. If plank is used for flooring, a common rule is to make the thickness of the plank one inch for every foot or fraction thereof in the spacing of the joists. If a reinforced concrete floor be used, the economic spacing appears to be about 2 feet 8 inches.

In order to give all the beams an even and equal bearing on the abutments, a channel is placed on top of the wall and under the beam ends. This is called a wall channel. Railings, illus-

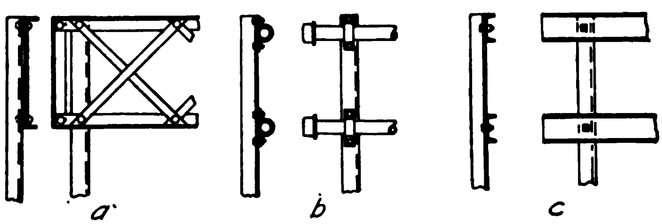


Fig. 149. — Details of Bridge Railings.

trated in Fig. 149, are placed on each side and supported by posts at intervals. Bridges of 15-foot spans or less should have two supports, one at each end. Bridges of spans exceeding 15-foot lengths should have in addition to the end supports a support at the one-third points or at the middle. Figure 150 gives the details of a beam bridge, while Fig. 151 shows the cross-

section. This is one of the American Bridge Company's standards somewhat modified by the writer, and may be taken as representing the best practice.

In case a post or support is put at the center of the span, allowance should be made for the holes taken out of the lower

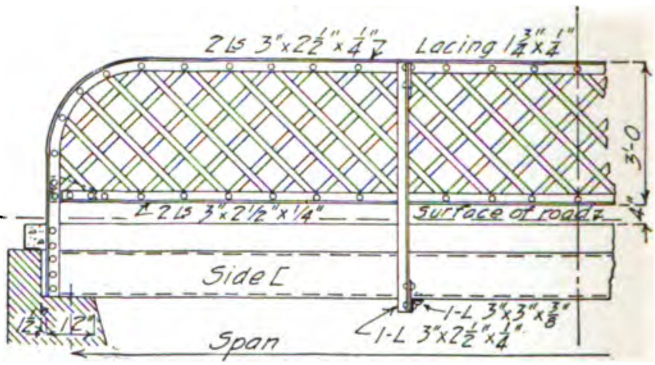


Fig. 150.—Side Elevation of a Beam Bridge.

flanges of the I-beams by the rivets which connect the cross-strut, or the strut should be connected to the I-beams by clips as in Fig. 151. A support in the center may be used without the strut underneath. In such cases the upright must be heavy

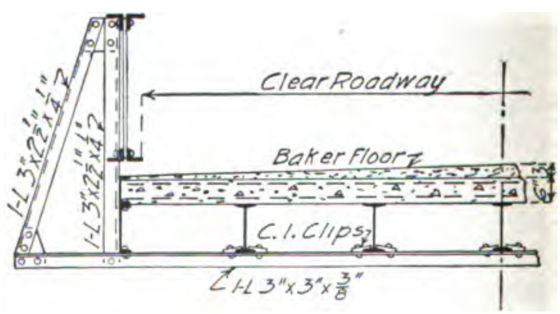


Fig. 151. —Cross-Section of a Beam Bridge.

and the beams connected at this point by gas-pipe separators at the neutral axis (see Fig. 152). The side channel, having a

large excess of area, will not be dangerously reduced in strength by having the necessary rivet holes taken from its section at the center where the rail support and laterals are connected to it. The dimensions given in Figs. 149 to 152 are common for all spans.

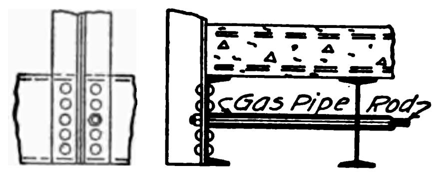


Fig. 152. — Detail of Center of Rail Support.

The sizes and weights of beam bridges for a 16-foot roadway and for a 15-ton engine or 125 pounds per square foot of floor service are given in Table I.

TABLE I. SIZES AND WEIGHTS OF BEAM BRIDGES.

SPAN, FEET.	PLANK FLOOR.			REINFORCED CONCRETE FLOOR.		
	5 I's.	2 ['s.	Wall Pls. Rivets, and Bracing.	5 I's.	2 ['s.	Wall Pls. Rivets, and Bracing.
8	7″ × 15-lb.	7″ × 9¾-lb.	590 lbs.	7″ × 15-lb.	7″ × 9¾-lb.	590 lbs.
12	7″ × 15-lb.	7″ × 9¾-lb.	600 lbs.	8″ × 18-lb.	8″ × 11¼-lb.	600 lbs.
15	8″ × 18-lb.	8″ × 11¼-lb.	620 lbs.	9″ × 21-lb.	9″ × 13¼-lb.	620 lbs.
20	9″ × 21-lb.	9″ × 13¼-lb.	630 lbs.	12″ × 31½-lb.	12″ × 20½-lb.	630 lbs.
25	10″ × 25-lb.	10″ × 15-lb.	640 lbs.	15″ × 42-lb.	15″ × 33-lb.	640 lbs.
30	12″ × 31½-lb.	12″ × 20½-lb.	650 lbs.	18″ × 55-lb.	18″ × 55-lb. I	650 lbs.
35	15″ × 42-lb.	15″ × 33-lb.	705 lbs.	20″ × 65-lb.	20″ × 65-lb. I	705 lbs.

The spans given in the table are center to center of bearings. The details of the bridges are as shown in Figs. 150 and 151, although any of the railings of Fig. 149 may be used. If the railing used is the one shown in Fig. 150, its weight may be taken as 20 pounds per linear foot of railing plus 45 pounds for each railing support. For example, if the bridge is 32 feet out to out and has railing supports in the center as well as at each end (see Fig. 150), the weight is $2 \times 32 \times 20 + 6 \times 45 = 1550$ pounds.

The weights of beam bridges of a 16-foot roadway with reinforced concrete floor and 15-ton engine capacity (exclusive of railings, railing supports, and lateral bracing) may be found by

$$w = 70 + 3.13\, l - 0.082\, l^2, \text{ for spans under 17 feet,}$$

$$w = 130 + 0.102\, l + 0.229\, l^2, \text{ for spans of 17 feet or over,}$$

in which w is the weight of steel in pounds per linear foot of span, and l is the span, in feet, center to center of bearings. These formulas are of little use except in the determination of the approximate dead load for spans other than those given in Table I. It is sufficiently accurate to take the weights given for that span which is nearest to the one which is to be designed. On account of the limiting sizes of manufactured beams, the limit of beam bridge spans is about 35 feet.

ART. 108. PONY TRUSS BRIDGES.

When the span lies between 30 and 80 feet, pony or low truss bridges are used. A pony truss bridge may be defined as a bridge whose height is such that top lateral bracing cannot be used. In practice the height is seldom equal to 12 feet.

Many short-span pony truss bridges have in the past been made with pin-connected Pratt trusses. Such bridges are lacking in lateral stability and are subjected to excessive vibration, giving rise to large impact stresses. Experiments made by the writer have shown impact stresses as great as 300 per cent of that caused by the static load. The bridges tested, and in fact any pin-connected pony truss bridges, are examples of bad practice, and should never be built.

The riveted Pratt truss (Fig. 153) or the riveted Warren truss are good types for low truss bridges. In case it is desired to use a reinforced-concrete slab floor with the latter type of truss, or to make the upper chord panels longer, the panel length

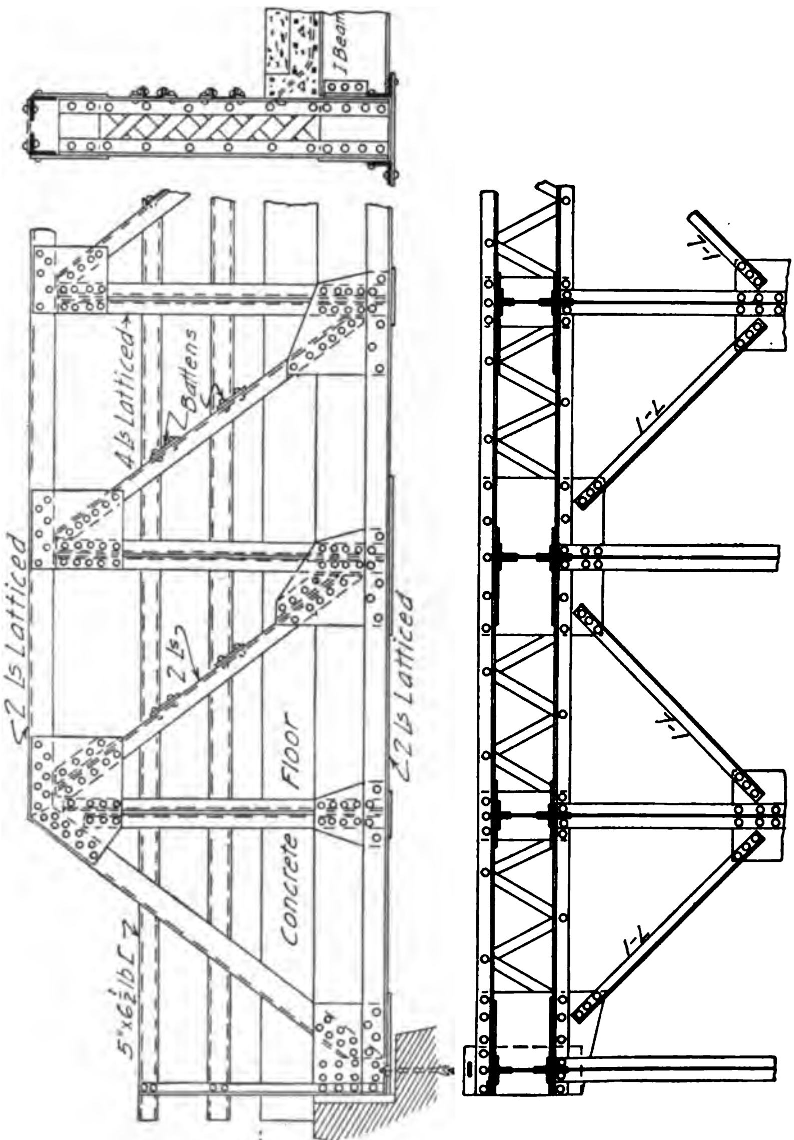


Fig. 153.— Riveted Pony Pratt Truss Bridge.

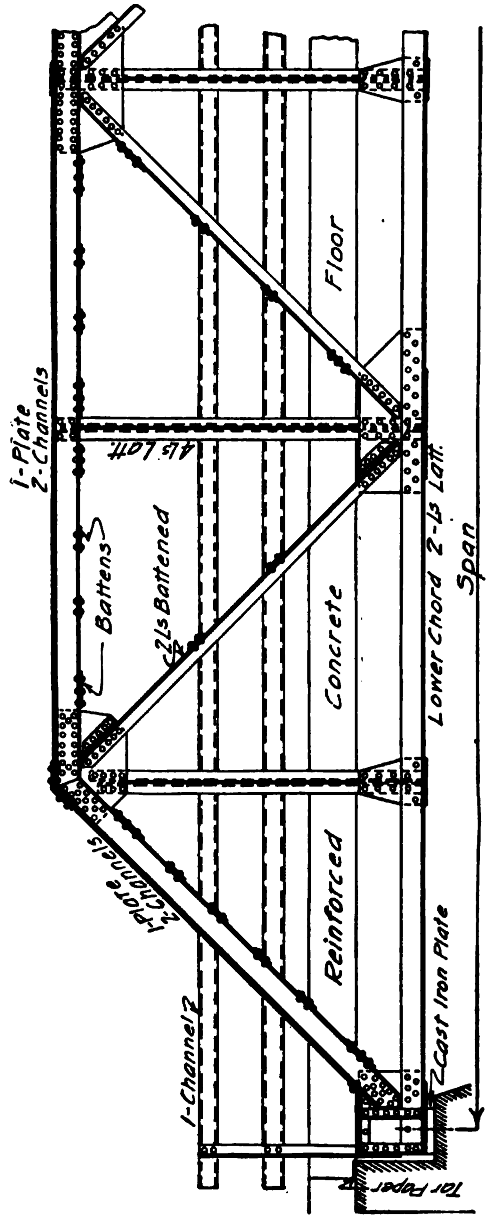


Fig. 154. — Riveted Pony Truss Bridge. Iowa State Highway Commission.

may be reduced by the use of subverticals, as in Fig. 154. Some low truss bridges have trusses with bent chords. This is not to be advised, since at places where the chord changes direction a splice is almost a necessity, and a splice should never be made in the top chord unless it is as strong as the chord section at its strongest part, or unless the chord is braced at that part.

The top chord should be braced at certain intervals along its length, preferably at every panel point. Unless this is done, great care should be taken in the design to see that it is safe about a vertical axis when considered as a column whose length is equal to the distance from hip to hip. In this one feature lies the great objection to such bridges, since adequate attention is rarely paid to it. It is permissible to build low trusses without such braces as those described above, and at the same time to design the chord sections for a length of one panel, but only provided the verticals at the panel points have such a great excess of strength about an axis parallel to the center line of the span and are so securely riveted to the ends of the floor beams that they act as braces to the top chord as well as vertical posts in the truss. In Fig. 155 are shown the details of two vertical posts, but (*a*) is far too weak to prevent any lateral deflection of the top chord. In (*b*) the pairs of angles are spread apart, and the post is of sufficient strength about the axis *1–1* to act as a brace for the top chord. In addition to other advantages of a concrete floor,

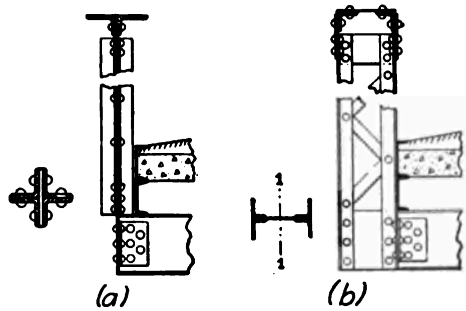


Fig. 155.— Details of Vertical Posts for Low Trusses.

it is seen that here it strengthens the post by having the side channel riveted to the post. It should be noticed that the chord is in itself greatly strengthened by the angles being spread, and this excess of strength about the vertical axis adds an additional element of safety to the truss. In case braces are desired, they

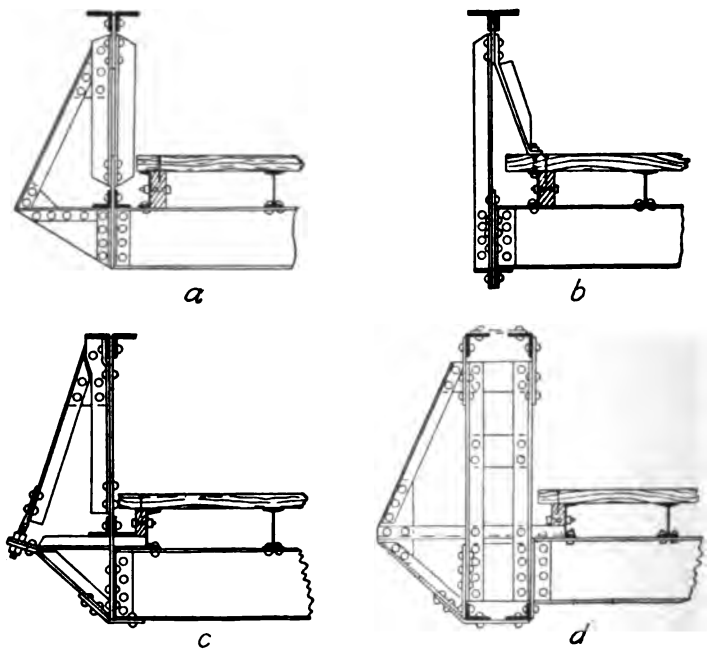


Fig. 156. — Details of Braces.

may be made as illustrated in Fig. 156. The writer advises the use of the detail shown in Fig. 156*d*. This not only has the spread angles vertical and a wide chord, but has the brace in addition.

Railings of various types may be used. Their section and the method of attaching them to the posts is shown in Fig. 153. As a rule, these bridges are bolted to the bridge seat with anchor bolts $\frac{3}{4}$ inch in diameter, set 12 inches in the masonry.

Slotted holes 2 inches long are cut in the bearing plates of the shoes at one end in order to permit expansion due to change of temperature.

The weights of the trusses and lateral systems, joists and floor beams, for a 16-foot roadway and 100 pounds per square foot of roadway for trusses and a 15-ton engine for the floor systems are given in Table II. If the joists are spaced more than two feet, the maximum bending moment which is caused by any single concentrated load on the floor is assumed to be taken by three joists.

TABLE II. DATA AND WEIGHTS OF RIVETED PONY TRUSS BRIDGES.

Span. Feet.	Plank Floor.				Reinforced Concrete Floor.			
	Height. Feet.	Wt. of Two Trusses and Lateral Bracing. Pounds.	Wt. of Stringers and Floor Beams. Pounds.	Wt. of Trusses per Linear Foot of Span. Pounds.	Height. Feet.	Wt. of Two Trusses and Lateral Bracing. Pounds.	Wt. of Stringers and Floor Beams. Pounds.	Wt. of Trusses per Linear Foot of Span. Pounds.
36	6	4 100	5 448	114	6.5	8 890	8 110	247
40	5.4	4 960	6 761	124	7	10 380	8 400	259
42	6	5 090	6 956	121				
50	6	7 000	8 750	140	7	13 250	11 500	271
50					7	13 830	11 040	271
60	6	9 600	10 400	174	8	19 980	12 710	223
60					10	18 800	12 300	
70	7.4	12 350	11 960	177	9	24 275	15 425	347
72	8.5	12 660	12 350	176				
75	8.5	13 010	12 740	174	9	27 590	16 380	369

The weights of the trusses of pony truss bridges with reinforced concrete floors, a 16-foot roadway, and a capacity of 100 pounds per square foot of roadway or a 15-ton engine (railing

and lateral bracing included), may be determined by the formula:

$$w = 225 + 0.36\,l - 0.01\,l^2 + 0.00041\,l^3$$

in which w is the weight of steel in pounds per linear foot of span, and l is the span in feet center to center of bearings. In order to get the total weight of the bridge, w must be multiplied by the span of the bridge, and to this must be added the weights of the stringers and floor beams, and of the reinforced concrete floor and earth cushion.

ART. 109. PLATE GIRDER BRIDGES.

Plate girder highway bridges have been used to a limited extent. They weigh about the same as pony truss spans of equal length and are somewhat less expensive. They are good in appearance and, when used in the through form, are especially useful, since the sides act as a railing (see Fig. 157). The great

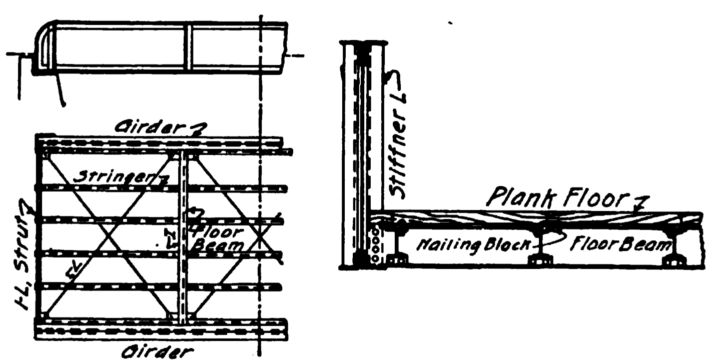


Fig. 157. — Through Plate Girder Bridge.

objection to these bridges is the lack of facilities for hauling them from the railroad station to the bridge site and for erecting them. In most cases of highway bridge building the material is hauled by local people, and the erection is done without other machinery than a hand winch, the members being light enough. The plate

girder is too heavy to handle in such a manner unless it is shipped in sections which are riveted together after being put in place on the falsework. This can easily be done, but the cost would be too great if done properly. If it is not done properly, the safety of the structure is too greatly impaired to allow its use. Plate girders for highway bridges are to be encouraged whenever facilities for their transportation and erection are available.

ART. 110. HIGH TRUSS BRIDGES.

When the span of a bridge becomes 80 feet or over, the trusses are made of such a height that the top chords may be connected by a lateral system. This is known as a high truss bridge. High trusses may be so arranged as to form a deck bridge, but such cases are rare in highway work. Theoretically, considering the sub- and superstructure only, the deck bridge is far more economical than the through, since an amount of masonry equal in volume to the cross-section of the piers times the height of the truss is saved, and a like saving is made at the abutments. Probably 99 per cent of all highway bridges are of one span and therefore have no piers, and the abutments reach up to grade in order to support the path of the approach. One great objection to the use of deck spans is that they cut down the opening for the stream to flow through; and in nearly all cases the waterway is not too large even with the through truss.

High truss bridges have been built for spans as short as 75 feet, but this is not to be recommended. Such a span weighs the same as a low truss bridge, but on account of the small sections of the members considerable vibration takes place.

The height of high truss bridges, center to center of chords, should not be less than 17 feet. Specifications call for a clear height of 13 feet, and 17 feet will allow for the thickness of the floor system and a good portal. Parallel chord trusses of the

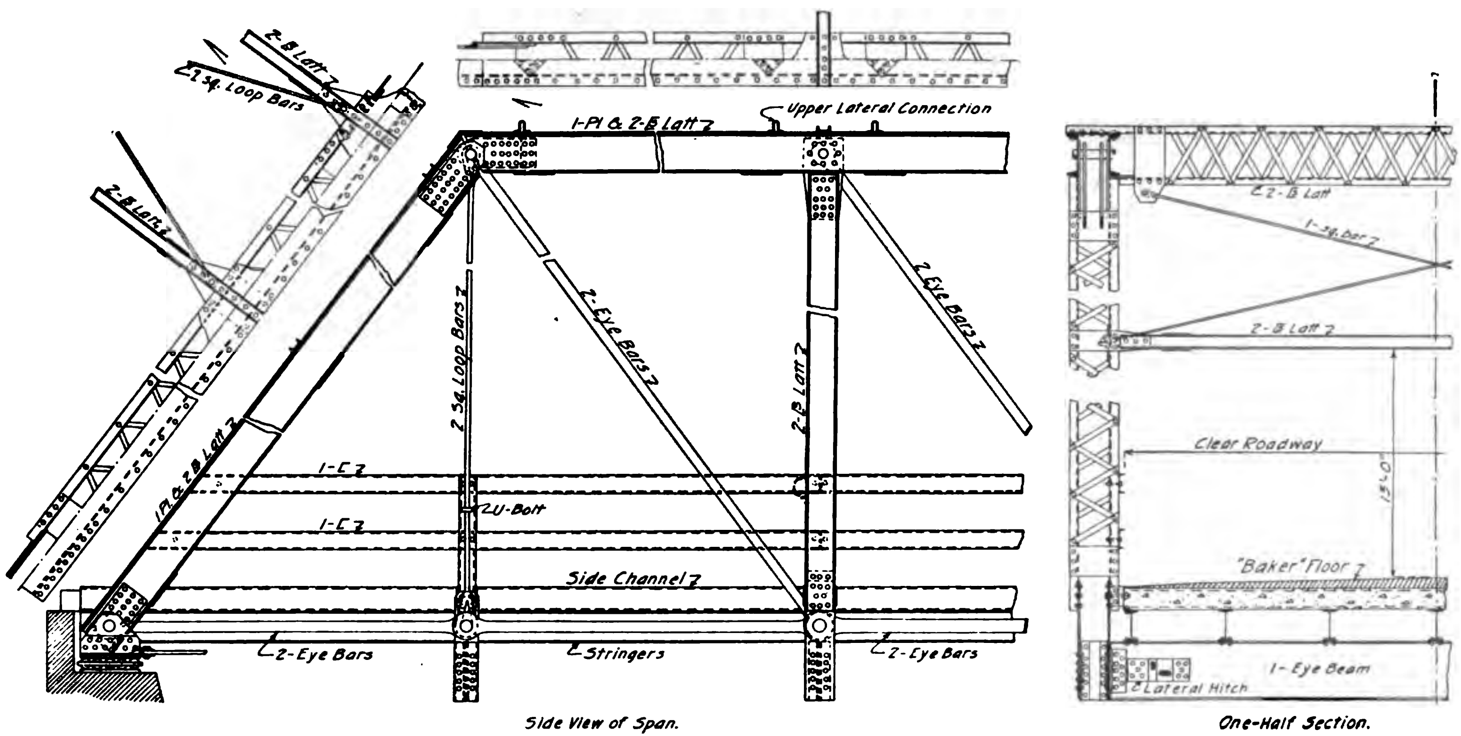
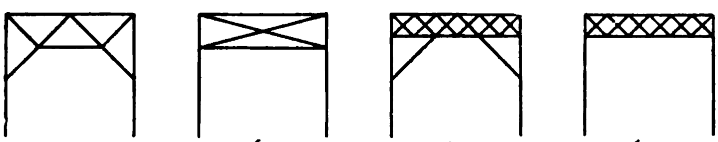


Fig 158. — Details of a Pin-Connected High Truss Bridge.

Pratt type are the most economical for spans up to 160 feet. Beyond this limit trusses with curved chords and subverticals should be used.

Riveted high trusses are seldom used for highway traffic. Their use for short spans when plank floors are employed is certainly advisable, but on account of the greater cost there is difficulty in persuading boards of supervisors or highway commissioners to use them. With the reinforced concrete floor sufficient weight is obtained and lateral stability secured so that riveted high trusses for short spans are neither necessary nor advisable.

Figure 158 shows the details of a high truss span. In many cases, especially where reinforced concrete floors are used, the lateral diagonals may consist of rods instead of angles. In case the span is 120 feet or less, the portals and intermediate cross-bracing should be of the form shown in Fig. 159*a*. In



(a) (b) (c) (d)

Fig 159. — Types of Portals and Intermediate Cross-Bracing.

case the span is greater than this, the portal and cross-bracing should be of the form in Fig. 159*b*.

The stresses in trusses with spans of 160 feet or less will be such that two channels may be used for the vertical post sections, two channels or two channels and a plate for the end post and top chord sections, and I-beams for the floor beams and joists.

The tension members should consist of eyebars, not loop bars, except in the case of the hip verticals, the first two panels

of the lower chords of bridges with plank floors, and the counters. Loop bars of steel should not be allowed except in counters, and here eyebars should be used if the section is large enough to permit their manufacture. Pins to which counters are attached should never be less than $2\frac{1}{2}$ inches and are seldom greater than 4 inches in diameter for spans up to 160 or 170 feet. This diameter of pin requires that the bar be not less than $\frac{3}{4}$ inch thick and 3 inches wide. Such a bar requires a section of $3 \times \frac{3}{4} = 2\frac{1}{4}$ square inches. If the counter is less in section than this, loop bars of soft steel, not medium steel, may be used provided their section is increased 25 per cent over that theoretically required, since experiments show that steel welds develop only about 75 to 85 percent of the strength of the bar. Welds of medium steel are not to be relied upon, since they cannot be made with any degree of certainty.

The use of bolts in the field connections of highway bridges is allowable if the holes are reamed and the bolts turned to a driving fit. In case the bolts are not turned to a driving fit, they may be used provided their number is 25 percent greater than that theoretically required, the excess being an allowance for a portion which are liable to be loose. In all cases the bolts should have the nuts screwed up tight.

Field rivets instead of bolts are required by most specifications. If these are hand driven, they are not to be preferred to bolts unless close inspection is given to them. Power driven field rivets are preferable to all other means of making field connections and should be employed whenever possible.

Pins less than $2\frac{1}{2}$ inches and rollers less than 3 inches in diameter should not be used. The rapid deterioration of pins of smaller diameter, and the liability of smaller rollers to become clogged with dirt so as to prevent rolling, are the principal

reasons why their use should not be encouraged, even if allowed theoretically.

The use of built-up pedestals and roller rests has been discontinued by many engineers. In their place is a rocker bearing of cast steel or cast iron; and this, at the free end, acts as a roller rest as well as a pedestal. In Fig. 160 are shown the details of the bearings for a 135-foot span with a 22-foot roadway, and reinforced concrete floor. They were designed by J. E. KIRKHAM, Bridge Engineer for the Iowa State Highway Commission, to stand a reaction of 103 000 pounds. This indicates that the allowable load per linear inch of rocker is 4900 pounds, whereas in accordance with COOPER'S specifications it may be as great as $300 \times 18 = 5400$ pounds per linear inch. In this equation 18 is the diameter of the roller or double the radius of the rolling surface. It is true that COOPER'S allowance is for medium steel rollers, but since the cast-iron rollers of Fig. 160 have given excellent service for many years the same allowance may be made for them as for medium steel rollers.

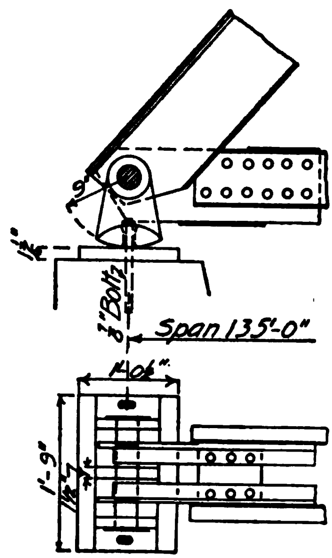


Fig. 160. — Detail of Cast End Bearings.

The cost of the built-up and of the cast pedestal is about the same. The advantage of the rocker bearing lies in the fact that it seldom becomes clogged, being almost self-cleaning. The rolling motion is not prevented by small pieces of gravel or by dirt and ice as is the case with rollers of small diameter, the large diameter allowing the rocker to pass over them, pressing

them flat. This form is easily cleaned. For the reasons given above this detail is to be recommended for end bearings.

Highway bridges of single spans seldom have end floor beams, the stringers in the end panels having their free ends resting upon the abutment either directly or indirectly. One of the simplest methods is to have the joists extend into the backwall, where they rest upon plates built in. The concrete of the backwall is prevented from adhering to the joists by the use of a thick coat of heavy oil or a wrapping of building paper during construction. This detail is shown in Fig. 161. It is the same for both the fixed and the free ends of the span with the exception that slotted holes are cut as shown. It is much cheaper than most details for this place, and has given excellent service in bridges where it has been in use for several years.

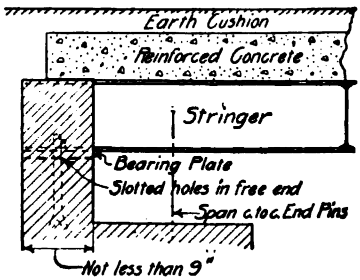


Fig. 161. — Backwall Bearings for End Stringers.

Many other methods are used to provide bearings for end stringers. Several of the best are shown in Fig. 162. In 162*a*

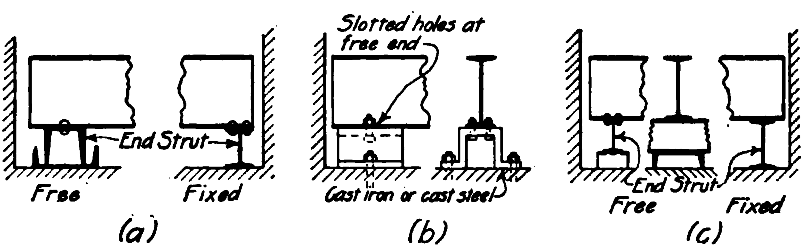


Fig. 162. — Stringer Bearings at Ends of Span.

and 162*c* the end strut runs from one shoe to the other, at the free end being connected to the pedestal above the roller rest

and at the fixed end to the base plate or pedestal. In all free end details the end strut moves with the movable shoe. For the rocker detail of Fig. 160, the bearing of Fig. 161 or of Fig. 162*b* is recommended for use.

Floor beams should be connected below the pins, since this reduces the distance from the bottom of the shoes to the bottom of the stringers, and this decrease makes the connection between the end stringers and the abutment less expensive. Nothing is gained by attaching the floor beams above the pins, and the practice is not recommended. The floor beams should be riveted to the trusses below the center line of the pins.

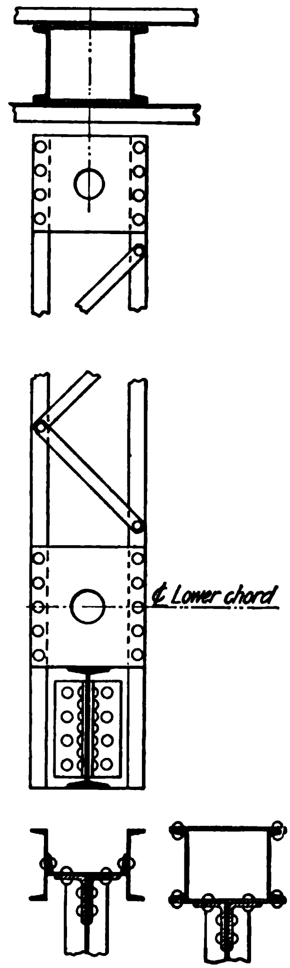


Fig. 163.—Detail of Vertical Post.

Since probably 90 per cent of all high truss bridges have trusses of the Pratt type, this one will alone be considered. There are two general types of vertical posts, namely: those with the web of the channels parallel to the center line of the truss, and those with the web of the channel perpendicular to the center line of the truss. When intermediate bracing of the type shown in Fig. 159*d* is used, neither type has any theoretical superiority over the other on account of the fact that the radius of gyration of the post does not have to be greater than the radius of gyration of one channel when taken relative to an

axis perpendicular to its web. The details of posts which have the channel webs parallel to the center line of the truss are shown in Fig. 158. The details of the other type are shown in Fig. 163. The practical advantages are all on the side of the type of Fig. 158, since better connections can be made at both top and bottom. In cases where cross-bracing of the types shown in Fig. 159, *a*, *b*, and *c* are used, the type of Fig. 158 becomes a necessity, since by its use a radius of gyration may be increased by spreading the channels, and therefore the stresses due to the bending caused by the action of the wind on the top chord can be taken care of with a much lighter channel than would be required by the other type. In case the design of the truss requires the counter to be a single bar or requires a tension member to be packed inside of the post connection at the pin, a hole must be made in the web of the channel, which is expensive. The type of Fig. 163 has one

TABLE III. WEIGHTS OF HIGH TRUSS HIGHWAY BRIDGES WITH PLANK FLOORS.

Span. Feet.	No. of Panels.	Height of Truss. Feet.	Total Weight of Bridge, exclusive of Stringers and Floor Beams. Pounds.	Total Weight of Stringers and Floor Beams. Pounds.	Weight per Lin. Ft. of Bridge, exclusive of Stringers and Floor Beams. Pounds.
75	5	17	14 600	11 440	195
80	5	17	15 300	12 210	192
90	6	18	17 850	13 860	198
100	6	20	20 600	15 200	206
110	6	21	23 753	16 840	216
120	7 or 8	22	27 797	18 740	232
130	7 or 8	23	33 500	20 100	258
140	8 or 9	24	39 140	22 140	279
150	9	25	45 700	23 500	305
160	9	27	52 700	27 330	329
170	9	28	58 900	29 840	347

advantage, since it allows the connection of the floor beam to be made at the axis of the post. This is not so great an advantage, however, as to make this type one which can always be recommended in actual practice.

The weights of high truss bridges with plank floors of equal spans vary with the width of roadway and the loading. For a 16-foot plank-floored roadway, a loading of 100 pounds per square foot of roadway for the trusses, and of a 15-ton engine for the floor and its connections, the weights are given in Table III. To the above weights must be added the weight of the railing. This can easily be computed from the details shown in Fig. 149.

High truss bridges with a 16-foot roadway, reinforced concrete floors, stiff lateral and transverse bracing, and having the stringers designed by assuming that the total weight of the 15-ton engine is distributed over an area equal to that covered by its wheel base, have the weights given in Table IV. The live load for which the trusses and floor beams were designed is 100 pounds per square foot of floor area.

TABLE IV. WEIGHTS OF HIGH TRUSS HIGHWAY BRIDGES WITH STIFF LATERAL BRACING AND REINFORCED CONCRETE FLOORS.

SPAN. FEET.	NO. OF PANELS.	HEIGHT. FEET.	WEIGHT OF TRUSSES AND BRACING.	WEIGHT OF STRINGERS AND FLOOR BEAMS.	TOTAL WEIGHT. POUNDS.	WEIGHT OF TWO TRUSSES AND ALL BRACINGS PER LIN. FT.
75	5	17	27 500	17 050	34 600	367
80	5	17	30 140	18 360	48 500	377
90	6	18	35 710	19 890	55 600	397
100	6	20	41 950	22 270	64 220	420
120	7 or 8	22	57 690	26 310	84 000	480
140	8 or 9	24	73 300	31 440	104 740	525
160	9	27	91 730	37 870	129 600	573

To the weights as given in the table the weight of the railings must be added.

The weights of two trusses, the lateral and transverse bracing and railing, for any span other than those given in Table IV may be calculated with the help of the formula

$$w = 262 + 0.96\,l + 0.006\,l^2,$$

in which w is the weight of steel, per linear foot of span of the bridge, floor beams and stringers excluded, and l is the span in feet, center to center of bearings.

In case the lateral bracing of the bridge consists of rods, the hip vertical of eyebars, and the floor is designed by considering that one-third of the bending moment caused by a 15-ton engine is taken by one stringer, the weight of the trusses will be less and the weights of the floor beams and stringers will be greater than the values given in Table IV. Table V gives data and weights of bridges so designed.

TABLE V. WEIGHTS OF HIGHWAY BRIDGES WITH RODS FOR LATERAL DIAGONALS AND WITH REINFORCED CONCRETE FLOORS. CLEAR ROADWAY 16 FEET.

Span. Feet.	Height of Truss. Feet.	No. of Panels.	Weight of Steel Exclusive of Floor Beams and Joists.	Weight of Floor Beams and Joists.	Total Weight. Pounds.	Weight of Steel Exclusive of Floor Beams and Joists per Linear Foot.
80	16	5	21 525	24 943	46 468	269
120	24	8	43 311	32 388	75 699	361
144	24	9	59 930	34 344	94 274	417
160	24	9	80 430	44 924	125 354	502

For other spans than those given in Table V the weight of steel exclusive of stringers and floor beams may be obtained from the formula

$$w = 230 - 0.75\,l + 0.0153\,l^2,$$

where w is the weight of steel, exclusive of floor beams and joists, per linear foot of span, and l is the span in feet, center to center of bearings.

Floor beams for bridges with a 16-foot roadway and a capacity of 100 pounds per linear foot or a 15-ton engine, the floor being of the JOHNSON or BAKER type, should have the sizes given in Table VI.

TABLE VI. SIZES AND WEIGHTS OF FLOOR BEAMS. CLEAR WIDTH OF ROADWAY 16 FEET.

PANEL LENGTH. FEET.	SIZE OF FLOOR BEAM.	TOTAL WEIGHT. POUNDS.
7½	15-inch × 42-lb.	930
8 to 15	15-inch × 55-lb.	1040
16 to 23	20-inch × 65-lb.	1280

Floor beam connections and lateral hitches will have from 15 to 16 percent of the weight of the beam itself.

The spacing of joists is a factor in determining their size and weight. The joists are usually spaced 2 feet, or 2 feet 8 inches. The latter spacing is the most economical. Table VII gives the sizes of joists for floors of the JOHNSON or BAKER type and a 15-ton engine capacity. Spacing 2 feet 8 inches.

TABLE VII. SIZES OF STRINGERS.

PANEL LENGTH. FEET.	SIZES OF STRINGERS.
8 to 11.9	7-inch × 15-lb.
12 to 15.9	8-inch × 18-lb.
16 to 24.9	12-inch × 31.5-lb.
25 to 30	15-inch × 42.0-lb.

While the weights given in Tables I to VI are of value for a close estimate of the probable cost (see Art. 112), or for the

computation of the dead load, they are not to be recommended for use in bidding, because they are not itemized sufficiently, although they may be close enough otherwise.

ART. 111. HIGHWAY BRIDGE ABUTMENTS.

Highway bridge abutments may consist of concrete, reinforced concrete, steel, or of combinations of steel and concrete or timber. Concrete abutments should have a base equal to four-tenths of the height from the grade of the road to the base

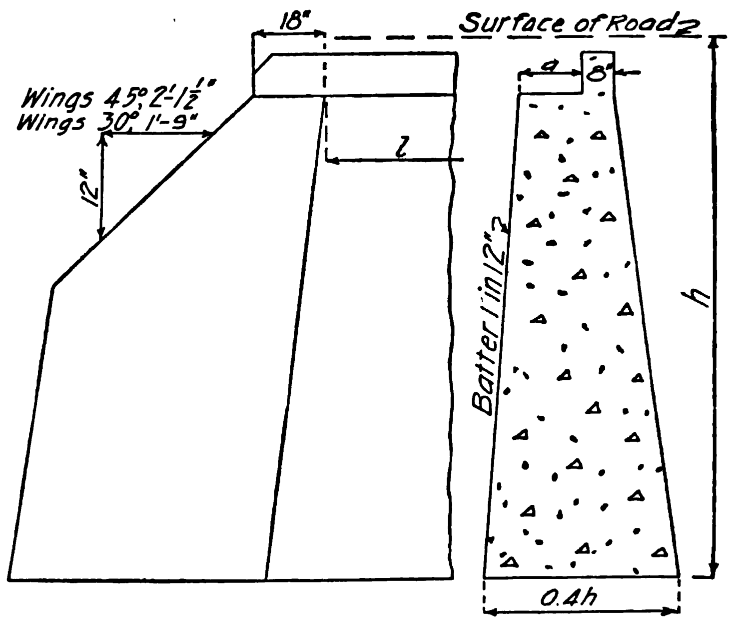


Fig. 164. — Concrete Abutment.

of foundation. The base of the foundation should be at least 3 feet below the bottom of the stream, unless solid rock or good hardpan is encountered before that depth is reached. Otherwise the excavation should be continued until a suitable foundation is reached, or piles should be driven. It will usually suffice

to have the wings make an angle of 30° with the face of the abutment. In some cases 45° will be better. If neither 30° nor 45° will satisfy the conditions, the wings should be given such a slope as to keep back the fill in the best possible manner. Figure 164 shows the standard form used by the writer, and Table VIII gives the necessary data for abutments for various spans. The upstream wings should slant back more than the downstream wings in case the approaches to the span are on a fill, the banks of the stream being low. An angle of 45° will generally be sufficient to protect the fill.

TABLE VIII. DIMENSIONS OF CONCRETE ABUTMENTS IN FIG. 164. CLEAR WIDTH OF ROADWAY 16 FEET.

SPAN OF BRIDGE IN FEET.	WIDTH OF BRIDGE SEAT, a.	LENGTH, l.
0 to 35	1 foot 4 inches	19 feet
35 to 50	1 foot 8 inches	19 feet
50 to 100	2 feet 0 inch	20 feet
100 to 150	2 feet 6 inches	22 feet
150 to 200	3 feet 0 inch	23 feet

The design of reinforced concrete abutments which resist overturning by their form alone is not within the scope of this article. For their design, the reader is referred to works on reinforced concrete design.

If the lower chord of the truss is of built-up members, as in riveted low truss bridges, or in case the structure is a beam bridge, the abutment may consist of a thin slab, no thicker than the backwall and bridge seat, reinforced on its outer face. Abutments less than 6 feet in height from the grade of the road to the surface of the ground do not require reinforcement, since their base, if equal to the thickness at the top, will be equal to or more than four-tenths of the height. A reinforce-

ment both horizontal and vertical of rods $\frac{1}{2}$ inch square spaced 12 inches on centers and placed $1\frac{1}{2}$ inches from the face will be sufficient for abutments up to 25 feet in height. The wing walls should be reinforced with the same sized rods spaced 2 feet between centers. Care should be taken to see that the base is sufficiently wide to give ample bearing area on the foundation. Figure 165 shows the detail of a base. In some cases it may be advisable to have counterforts located behind the bearing plates and have the wall

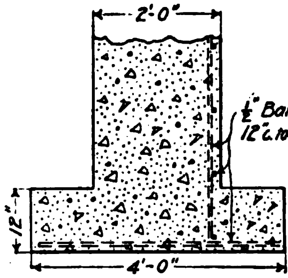


Fig. 165. — Detail of Base.

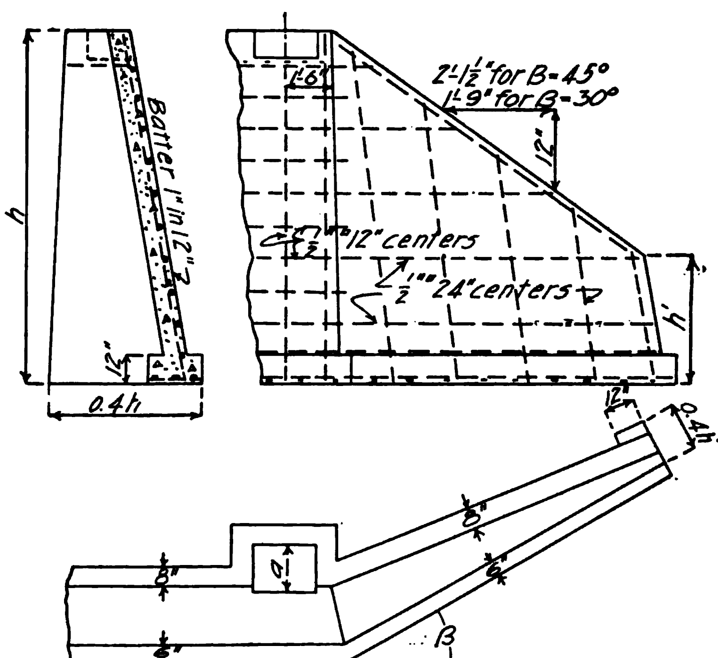


Fig. 166. — Detail of Abutment with Counterforts and Curtain Wall.

connecting them much thinner. Figure 166 gives the details of such an abutment. The value of a is given in Table VIII.

Care must be taken in all cases where the thrust of the abutment is partially resisted by the truss to compute the amount of thrust taken by the truss and to fasten the shoes to the bridge seat with bolts of sufficient section to resist the resulting shear. No allowance should be made for movement due to temperature. Both ends of the span should be fixed. Care should be taken to compute the stresses in the bottom chord due to the temperature and the thrust of the abutment. These stresses should be combined with the dead load stresses and the resultant stresses provided for in the design of the trusses. In every case the design should be compared with a plain concrete abutment and the costs of both determined and compared.

All-steel abutments usually consist of cylinders of steel placed upon a suitable foundation, filled with concrete, and connected by means of a curtain of plates and shapes. The wings to such abutments consist of shapes and plates riveted to each other, and to the piers, as indicated in Fig. 167. These wings are usually the chief source of weakness, although there are several others. The outer ends are often strengthened against being pushed over by earth pressure by having the channel at that point either driven deep into the ground or carried down and attached to a mud sill of either timber or steel. In either case the resistance to overturning is insufficient and the wings either fall entirely or tip over partially. Another source of weakness lies in the fact that the piers are seldom carried down far enough to prevent them tilting forward. In fact it is impossible, except at prohibitive expense, to carry them down sufficiently deep in ordinary earth to prevent their tipping forward. The beams which connect the two piers should be carefully designed to withstand the earth pressure which comes upon them.

These abutments have been used for all spans and all types of trusses. Their use for pin-connected spans results, in many cases, in the roller rest being pushed either partially or wholly

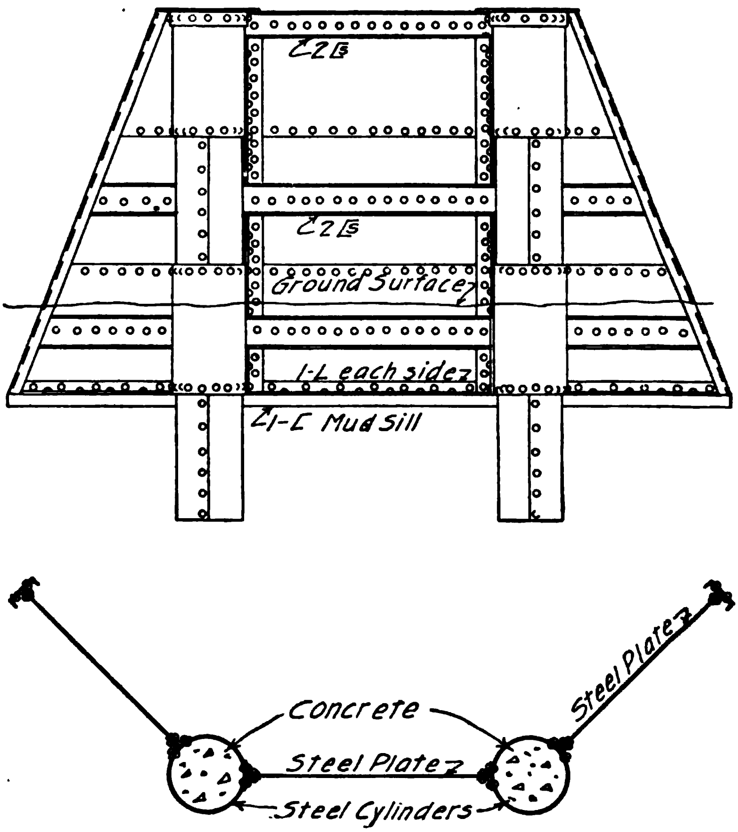


Fig. 167.—Cylinder Abutment.

out from under the pedestal, on account of the earth pressure tilting forward the abutment cylinders. Their use for trusses with riveted lower chords would be advisable were it not for the fact that no satisfactory method has yet been devised to prevent the wings from falling forward.

In beam or riveted truss bridges, the abutment frequently consists of vertical steel columns instead of the cylinders shown in Fig. 167. These columns are fastened securely to the bridge shoes at the top while at the bottom they are fastened to a mud sill of timber or steel. The space between the columns is usually filled with a concrete or timber backing, although steel plates may be used as in the case of cylinder abutments. The wings to such abutments may be made of a wood backing held in place by wooden piles, or it may be made as in the cylinder

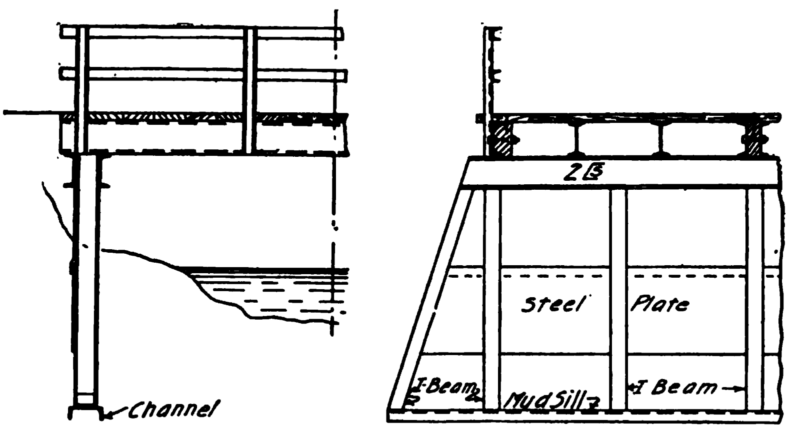


Fig. 168. — Leg or Bedstead Bridge.

abutment of Fig. 167. Bridges supported in this manner are called leg or bedstead bridges. Fig. 168 shows one with a steel plate backing. In many cases where the abutment is not high and the span is sufficiently long, the wings may be dispensed with and the fill allowed to fall forward or to be held in place by a cribbing of some kind.

Concrete in abutments and reinforced floors cost in 1910 from $7.25 to $8.50 per cubic yard; in the backing of bedstead bridges about $8.00 per cubic yard; and in reinforced concrete

abutments, including the rods, from $9.00 to $12.00 per cubic yard. Rods in reinforced floors cost about 2.7 cents per pound in place.

ART. 112. COST OF HIGHWAY BRIDGES.

The cost of a bridge varies with the price of the raw material, the freight haul, the length of haul from the railroad station to the site, the difficulties of erection, the profit desired by the manufacturers, and for spans up to 60 feet, with the shop cost. For spans over 60 feet the shop cost does not vary much. Table IX gives the cost per pound of material of the various items which combine to make the total cost. Manufacturer's profit is not included. The cost of material includes the freight of the raw material from the mill to the shop, the unloading, and storage until used. This rate of 2.00 cents per pound was figured when steel was 1.88 cents per pound. This shows that 0.12 cents per pound was the cost of freight, unloading, and storage. These costs were determined in 1908 at a plant having a capacity of 2500 to 3000 tons per annum.

TABLE IX. DETAILED COSTS OF STEEL BRIDGES.

All values are given in cents per pound.

Item.	Beam Leg Bridges. Spans 8–26 Feet.		Pony Truss Bridges. Pin-Connected. Spans 24–60 Feet.		Through Truss Bridges. Pin-Connected. Spans 82–140 Feet.	
	Range.	Aver.	Range.	Aver.	Range.	Aver.
Metal	2.00–2.00	2.00	2.00–2.00	2.00	2.00	2.00
Erection	0.40–1.10	0.68	0.46–1.43	0.78	0.52–1.06	0.78
Shop	0.14–0.40	0.26	0.35–0.73	0.51	0.47–0.56	0.52
Haul	0.11–0.41	0.22	0.10–0.36	0.18	0.04–0.14	0.09
Freight	0.10–0.50	0.23	0.08–0.39	0.24	0.05–0.39	0.19
Incidental	0.13–0.46	0.33	0.18–0.70	0.41	0.14–0.31	0.20
Total	3.20–4.38	3.72	3.42–4.82	4.12	3.43–4.18	3.78

The items in Table IX are stated below, and under them the sub-items which they include.

METAL

Raw Material
Freight to Shop
Unloading and Storing

ERECTION

Labor
Traveling Expenses
Boarding and Lodging
Minor Items such as Rails, Lost Tools, Telephone and Telegraph

SHOP

Labor
Heat
Light
Fuel, Oil, etc.
Power
Minor Items

HAUL

Loading
Hauling to Bridge Site
Unloading

FREIGHT

Freight from Shop to Bridge Site

INCIDENTAL

Office Work
Bidding Expenses
Drafting, Engineers' Expenses
Advertising
Shop Foreman
Minor Items

It will be noticed that the shop cost is quite constant for the through truss spans. In any particular case the freight can be determined exactly, the haul may be computed at the rate of 25 cents per ton-mile and 50 cents per ton for loading and unloading, the erection may be taken from Table IX or X or it may be estimated more closely in case a survey of the bridge site is available, the incidentals may be taken from Table IX, and a profit of from 10 to 20 per cent added.

In many cases it is customary to estimate the erection at so much per foot of span. Table X gives the cost of erection of highway bridges. Each value is the average for several bridges.

TABLE X. COST OF ERECTION OF HIGHWAY BRIDGES.

Beam Bridges.		Pony or Low Truss. Pin-connected.		High Truss Pratt. Pin-connected.	
Span in Feet.	Cost per Linear Foot.	Span in Feet.	Cost per Linear Foot.	Span in Feet.	Cost per Linear Foot.
8	$1.87	24	$4.75*	82	$2.25
10	2.42	30	2.77	85	2.08
14	2.33	32	1.65	100	2.94
16	1.70	36	2.36	120	4.48*
24	0.90	40	1.53	140	3.70
25	0.94	42	2.02		
26	2.17	45	1.43		
		56	1.36		
		60	2.15		
Average	$1.76		$1.91		$2.74

* Not included in the general average.

Tables IX and X give data for bridges erected in the prairie states of the Middle West, and their cost of erection is quite low when compared with hilly or mountainous regions, in which case, the cost may run up as high as $7.00 to $8.00 per linear foot of span.

At the present time (1910) the steel in highway bridges may be estimated at from 3.8 to 4.25 cents per pound with two coats of paint and erected in place ready for traffic. Painting may be estimated by considering that ½ gallon will give one ton its first coat while ⅜ gallon will be sufficient for the second coat. A common rule is to estimate one gallon per ton of material as giving two coats. A painter can apply two coats of paint on 1 to 1½ tons of steel in a day of 8 hours.

For example, let it be required to estimate the cost of the steel erected for a 160-foot span in the Middle West, the bridge site being 3 miles from the railroad station and at such a distance from the plant that the freight is 0.10 cents per pound.

From Table X:

Metal	2.00 cents
Shop	0.52 cents
Incidentals	0.20 cents
Freight	0.10 cents
Total	2.82 cents

From Table V the total weight is 125 354 pounds.

ESTIMATE OF COST.

Steel, 125.354 pounds @ 2.82 cents	$3534.98
Erection, 160 feet @ $2.75 (Table X)	440.00
Hauling, 63 tons × 3 miles × 25 cents	47.25
Loading and unloading, 63 tons @ 50 cents	31.50
Paint, 63 gallons @ $2.00	126.00
	$4279.73
Profit, 10 per cent	427.97
Probable cost ready for traffic	$4707.70

As a matter of fact the bids on the above steel work ranged from $4430 to $4950. One company agreed to furnish the material at the bridge site for 3.65 cents per pound; this would make the cost as follows:

Steel, 125.354 pounds @ $3.65	$4575.42
Erection, 160 feet @ $2.75	440.00
31.5 gallons of paint @ $2.00	63.00
	$5078.42

The erection of this bridge actually cost $1.80 per linear foot of span. The quotation of 3.65 cents at the bridge site is thus shown to be too high.

ART. 113. DATA FOR A HIGH TRUSS SPAN.

Let it be required to design a Pratt truss bridge, 160 feet in span, with a clear roadway of 16 feet, the floor being of the BAKER or JOHNSON type. The bridge will be designed under

COOPER's General Specifications for Steel Highway and Electric Railway Bridges and Viaducts (edition of 1909 revised by BERNT BERGER), with the exception: (*a*) that a live load of a 15-ton engine (see Fig. 144), or 100 pounds per square foot of floor surface will be used in designing the floor and its connections; (*b*) that 100 pounds per square foot of floor surface will be used as the live load in designing the trusses; (*c*) that §§ 4, 18, and 21, of the specifications are to be omitted; (*d*) that lateral rods and a reinforced concrete floor of the BAKER type are to be used; and (*e*) that in § 97 a thickness of $\frac{5}{16}$-inch metal will be allowed in the webs of channels in the lateral systems.

The weight of the steel and floor with its earth cushion is computed as follows:

Weight of steel as per Table IV		125 354 pounds
Floor and cushion, 160 × 16 × 125		256 000 pounds
	Total	81 354 pounds

According to Table V there should be 9 panels. This makes each dead panel load 381 354/(2 × 9) = 21 190 pounds. The live panel load is ½ (160 × 16 × 100/9) which, in round numbers, is 14 230 pounds. The wind panel loads are: for the top panel points 2660 pounds fixed and for the bottom panel points 2660 pounds fixed and 2260 pounds moving.

The dead, live, and wind load stresses are computed by the methods of Part I, and are recorded on the stress sheet in Fig. 169, the height of the trusses being 24 feet, according to Table V.

ART. 114. DESIGN OF STRINGERS

The dead load for the stringers consists of the weight of the concrete floor, the earth cushion and the weight of the stringers themselves. Since the stringers are spaced 2′ 8″ centers, the

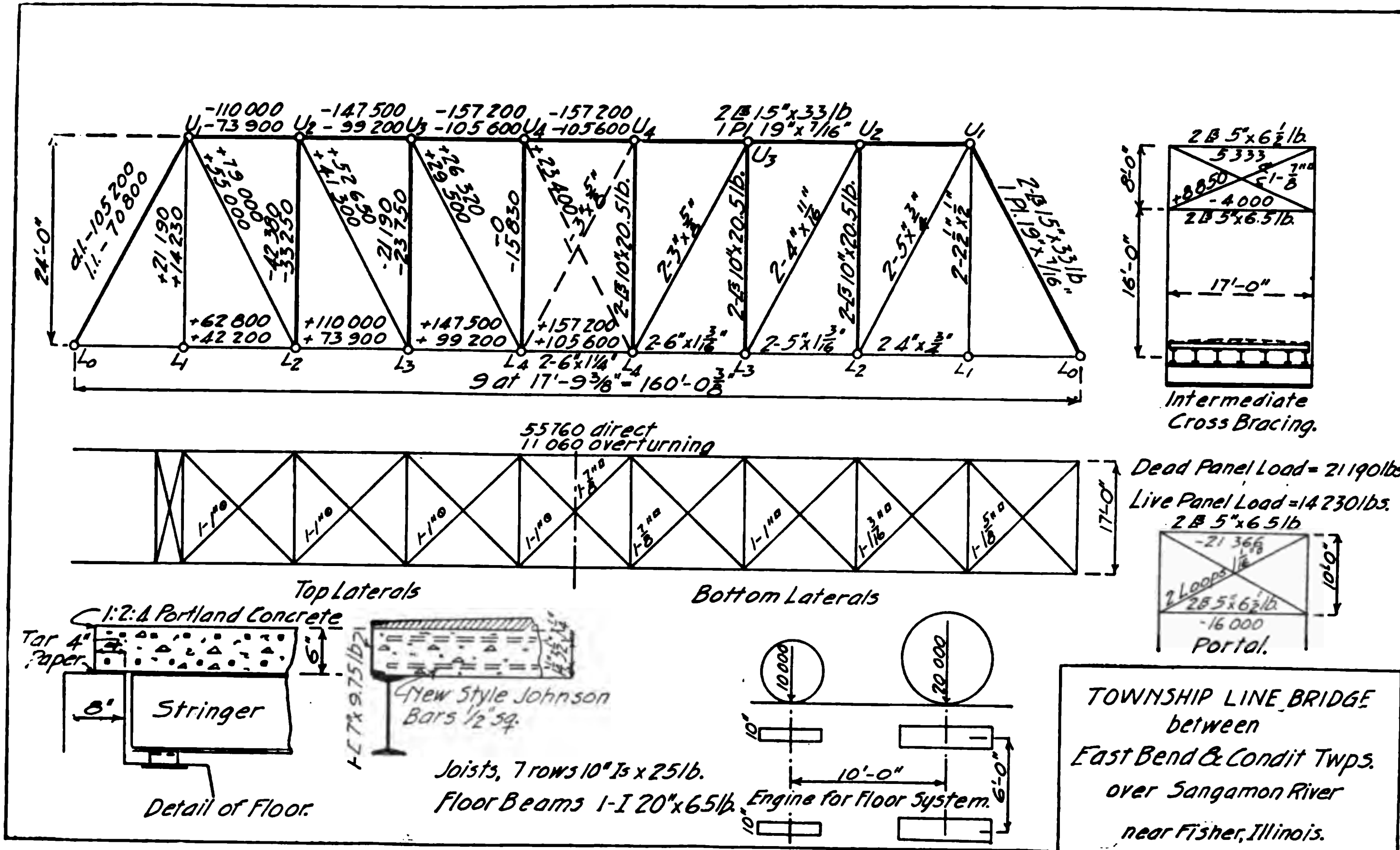


Fig. 169. Stress Sheet of Through Pratt Truss Highway Bridge.

floor and its cushion will weigh $2.67 \times 100 = 267$ pounds per linear foot. The weight of the stringers may be approximated by an inspection of Table VII. Each joist for a span of 17.78 feet weighs about $31\frac{1}{2}$ pounds per linear foot. The total dead load is now readily computed to be $267 + 31\frac{1}{2} = 298\frac{1}{2}$ pounds per linear foot.

The road roller, Fig. 144, for the computation of the live load stresses in the stringers does not give as large stresses as a farm or traction engine of equal weight. The 15-ton engine, which will be used in designing these stringers, has its wheel base and weights given in Fig. 170.

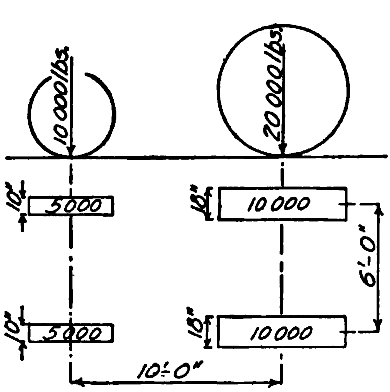


Fig. 170.—Wheel Base of 15-Ton Engine.

The panel length being 17.78 feet, the position of this engine for maximum bending moment will be such as to bring the heavy wheel in the center of the span, since the wheel spacing exceeds 0.586 times the span. The bending moment due to this wheel at that point is $\frac{1}{3} \times (10000/2) \times (17.78 \times 12/2) = 177\,800$ pound-inches. The dead load moment under the wheel is $(298\frac{1}{2} \times 17.78 \times 17.78 \times 12)/8 = 141\,500$ pound-inches and this together with the live load moment makes a total of 319 300 pound-inches. This requires a section modulus of $319\,300/13\,000 = 24.55$ inches^3. A 10-inch 25-pound I-beam satisfies this condition and will be used. This beam weighs a few pounds less than the assumed weight of the stringer, but since the difference does not cause a stress equal to 10 per cent of the dead and live load stresses, it is not necessary to redesign it.

ART. 115. DESIGN OF FLOOR BEAMS.

The floor beam span is taken equal to the clear roadway, or 16 feet. The effective span is in reality somewhat greater, but since the difference is seldom more than 6 inches, a length taken equal to the clear width of the roadway will not make any material difference in the design.

The joists are spaced so closely together that the dead load may be considered as uniformly distributed over the floor beam's span. The dead load is:

Floor and cushion $100 \times 16 \times 17.78$	$= 28\,448$ pounds
Stringers $7 \times 25 \times 17\ 78$	$= 3\,112$ pounds
Side channels $2 \times 12.25 \times 17.28$	$= 236$ pounds
Total dead load on floor beam	$= 31\,796$ pounds

To this must be added the weight of the floor beam itself. This, according to Table VI, may be taken as 55 pounds per linear foot, or $16 \times 55 = 1450$ pounds, which brings the total dead load for the floor beam up to $(31\,796 + 1450) = 33\,246$ pounds. The dead load bending moment is $(33\,246 \times 16 \times 12)/8 = 797\,904$ pound-inches.

The greatest bending moment will be caused either by the engine or by the uniform load of 100 pounds per square foot of floor surface. In order to get the greatest moment from the engine it should be placed so that the heavier wheels are directly over a floor beam and one of these wheels is at a distance from the center of the floor beam equal to one-fourth of the distance between the wheels. In this case the small wheel will have $(17.78 - 10)5000/17.78 = 2188$ pounds of its weight transferred to the floor beam. There will then be two loads of $10\,000 + 2188 = 12\,188$ pounds 6 feet apart on the floor beam, one of these loads being 1.5 feet from the left of the center of the floor beam and the other wheel on the opposite side of the center. The left reaction is 9900 pounds, and the moment under the wheel

nearest the center is $9900 \times 6.5 \times 12 = 772\,200$ pound-inches. No other load is assumed on the panel ahead, since it is extremely unlikely that any heavy load will precede or follow a traction engine. It is true that heavy separators may be attached to the engine, but their first wheel is usually so far away from the rear wheel of the engine as to be in the next panel, and hence will not stress the floor beam in question.

The bending moment occurring when a live load of 100 pounds per square foot of roadway is applied is $100 \times 16 \times 17.78 \times 17.78 \times 12 / 8 = 758\,700$ pound-inches. Since this is less than that caused by the engine, the engine moment will be used. The dead load moment at the same place is $[16\,623 \times 6.5 - (33\,246 / 16)\,(6.5^2/2)]\ 12 = 769\,800$ pound-inches. The total moment $772\,200 + 769\,800 = 1\,542\,000$ pound-inches, which requires a section modulus of $1\,542\,000 / 13\,000 = 118.6$ inches^3. A 20-inch 65-pound I-beam will be used (§162, Cooper), and for reasons stated above, a redesign will not be necessary.

The above numerical values and others in this chapter were obtained by the use of the slide rule, an instrument for performing multiplication and division where errors less than one-tenth of one per cent are disregarded.

ART. 116. DESIGN OF EYEBARS.

The eyebars should not be wider than four-thirds the diameter of the pin to which they are attached (§ 104, Cooper), and in general the thickness of a bar should not be less than one-sixth its width. Pins safe in bending are liable to be deficient in bearing. For this reason it is advisable to design the bars so that they will not be deficient in bearing on pins of minimum diameter. A relation satisfying this condition will now be deduced.

Let t be the thickness of the bar, W its width, P the total stress it is required to sustain, D the diameter of the smallest pin, and S the allowable unit bearing stress. Then $SDt = P$. But $D = \frac{4}{5} W$, and $W = 6t$. Substituting these values and reducing, $48\ St^2 = P$. Here $S = 15\,000$ pounds per square inch for live load, and hence $t = 0.00365\ P^{\frac{1}{2}}$ is the minimum allowable thickness of a bar. Guided by this, and knowing that bars under six inches should be ordered in variations of one-half inch and bars over six inches should be ordered in variations of one inch, it is now easy to find the sizes of the eyebars whose maximum stress is known. The maximum stress in the web members is readily found by adding one-half of the dead load stress to the live load stress (§ 45, Cooper). This sum divided by the number of bars which are to carry it, gives the load P in the preceding formula. The minimum thickness is then computed. Next the area and the maximum width are found, and lastly the final size. For web members the widths should generally decrease from the ends toward the middle of the truss. For lower chord members the widths should generally increase from the ends toward the middle. According to § 52 of the Specifications, the wind stresses need not be considered in L_4L_4 since $65\,820 > 0.3\ (157\,200 + 105\,600)$.

In case the wind stress had been $86\,000$, it would be necessary to take it into account. The maximum stress would then be $157\,200 + 105\,600 + 86\,000 = 348\,000$ pounds. The unit stress which, according to § 52, Cooper, would then be used as a divisor, is $23\,190$ pounds per square inch, and is determined as follows: Let L denote the live load stress, D the dead load stress, W the wind stress, S the allowable live load unit stress, S' the average allowable unit stress for both dead and live loads, A the area required for dead and live load stresses, and A' the area required for stresses due to dead, live, and wind loads.

Then $\quad A = L/S + D/2S$, and $S' = (L + D)A$,

whence $\quad S' = S(2L + 2D)/(2L + D) \qquad . \qquad (1)$

Cooper, § 52, requires that $A' = (L + D + W)/1.30S'$. On substituting the proper numerical values there is obtained

$$S' = 12\,500 \frac{2 \times 105\,600 + 2 \times 157\,200}{2 \times 105\,600 + 157\,200} = 17\,840 \text{ lb. per sq. in.,}$$

$$A' = (105\,600 + 157\,200 + 86\,000)/(1.30 \times 17\,840) = 15.04 \text{ sq. in.}$$

The preceding discussion is given here to illustrate the method of procedure in cases where the wind stresses have to be considered.

The number of bars to be taken for any member is a matter of choice in some respects. An even number should, of course, always be taken, except when only one is needed. They should be so chosen and packed that the flexure of the pin is a minimum. A large number of bars decreases this flexure, while a small number increases it. It costs almost the same to forge a large eyebar as it does a small one, while the manufacture of large pins is much more costly than the manufacture of small ones.

The problem resolves itself into this form, namely, that the cost of eyebars and pins shall be a minimum. The shop practice of different plants modifies the results obtained as to the numbers of bars which satisfy these conditions, and therefore no rigid rule can be given. The stress in the heaviest eyebar of this bridge due to the weight of the bar itself is readily computed to be 850 pounds per square inch, which need not be considered.

A table can now be formed as follows. One of the diagonals U_4L_4 is to be adjustable, turnbuckles being used, and the bars are to be upset according to § 105 of the Specifications.

MEMBER.	P POUNDS.	NO. OF BARS.	t IN INCHES.	UNIT STRESS. POUNDS PER SQUARE INCH.	AREA REQUIRED IN SQUARE INCHES.	MAX. WIDTH, INCHES.	BAR USED. INCHES.
L_0L_1	36 800	2 eye	0.701	12 500	2.94	4.20	$4 \times \frac{3}{4}$
L_1L_2	36 800	2 eye	0.701	12 500	2.94	4.20	$4 \times \frac{3}{4}$
L_2L_3	64 450	2 eye	0.927	12 500	5.15	5.56	5×1
L_3L_4	86 475	2 eye	1.073	12 500	6.90	6.44	6×1
L_4L_4	92 100	2 eye	1.109	12 500	7.40	6.65	$6 \times 1\frac{1}{4}$
U_1L_2	47 250	2 eye	0.792	12 500	3.78	4 78	$5 \times \frac{3}{4}$
U_2L_3	33 838	2 eye	0.672	12 500	2.72	4.05	$4 \times \frac{11}{16}$
U_3L_4	24 300	2 eye	0.569	12 500	1.72	3.03	$3 \times \frac{5}{8}$
U_4L_4	23 400	1 loop	0.559	12 500	1.88	3.36	$3 \times \frac{5}{8}$
U_1L_1	17 710	2 loop	0.485	8 000	1.22	2.52	$2\frac{1}{2} \times \frac{1}{2}$

ART. 117. DESIGN OF POSTS AND PINS.

POST U_2L_2. The posts will consist of two channels laced, their webs being parallel to the plane of the truss. From § 48 of the Specifications it is noted that the radius of gyration cannot be less than $24 \times 12/100 = 2.88$ inches, and according to § 97, Cooper, the thickness of the metal must not be less than $\frac{5}{16}$ inch. A 10-inch 20-pound channel satisfies these conditions, and will now be investigated.

The maximum stress in this post reduced to equivalent live load stress is $42\,380/2 + 33\,250 = 54\,440$ pounds. The unit load allowed by the Specifications is $P = 10\,000 - 45\,\frac{l}{r}$, where l is the length in inches. Here $l = 288$ inches, $r = 3.66$ inches, and the area $A = 2 \times 5.88 = 11.76$ square inches. Then $P = 10\,000 - 45 \times 288/3.66 = 6460$ pounds per square inch, whence $54\,440/6460 = 6.44$ square inches is the section area required. Thus, it is seen that in order to meet the conditions of § 48 and § 97 of the Specifications, the area is greatly in excess of that required by the given formula. A 9- or 10-inch 15-pound

channel could be used if it were required to satisfy the conditions for unit load only.

WIND ON THE VERTICAL POSTS. — The channels for these posts should be placed a certain distance from back to back, which will not only insure safety against the compression of the vertical load, but also that due to the effect of bending at the point where the transverse wind bracing is connected. This latter bending is caused by the wind.

Assuming that a 4-inch pin will be used at panel point L_4, the head of the 6-inch eyebar will be found to be 12 inches upon consulting the eyebar tables in manufacturers' handbooks. Allowing a clearance of 1 inch between the top of the floor beam and the eyebar head and using a BAKER floor, the distance from the center line of the pins to the top of the floor is 12″, as shown in Fig. 171. This causes the lower cross-bracing strut to join the vertical post at a point $15+1=16$ feet above the center line of the bottom pins. The bending moment caused by the wind (see Fig. 172) is $1333\frac{1}{3} \times 16 \times 12 = 256032$ pound-inches, since the posts cannot be assumed to be fixed at the lower end. In reality no bending would come upon the post due to wind if the wind acted only on the top chord, since in this case the top lateral bracing would carry the wind reactions to the portals. Since part of the wind panel loads of the top chord are due to wind blowing on the post and tension members, it is customary to consider only a portion of the 2667 pounds to

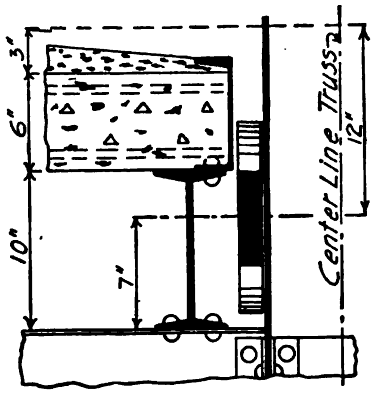


Fig. 171.

cause bending in the post. One-half is often chosen arbitrarily, although the writer considers that one-third would be more nearly correct. In this case one-half will be taken. This leaves a wind moment of $256\,032/2 = 128\,016$ pound-inches to be provided for about an axis parallel to the webs of the channels.

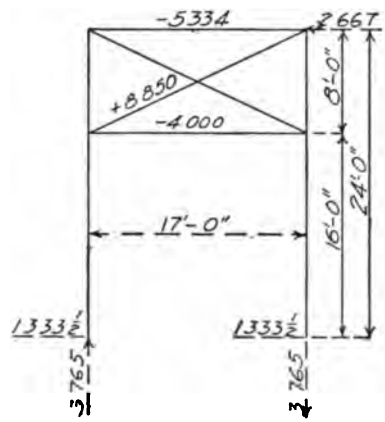


Fig. 172.

Since the post is of larger section than necessary, the section will be assumed to be such that the radius of gyration is the same with respect to both axes and will then be investigated. According to the manufacturers' handbooks the channels must be 5.97 inches back to back if flanges are turned out as in this post. The general formula for combined bending and compression is (Mechanics of Materials, Art. 117),

$$S_1 = Mc/\left(I - \frac{P}{k}\frac{l^2}{E}\right).$$

Here k should equal 10 when both the ends are free, 24 when one end is free and one end is fixed, and 32 when both ends are fixed. In all cases P equals the total direct compression in the member. For the case in hand $M = 98\,016$ pound-inches, $c = 5.97/2 + 2.74 = 5.73$ inches, $I = 2 \times 78.7 = 157.4$ inches^4, $P = 42\,380 + 33\,250 = 75\,630$ pounds, $l = 24 \times 12 = 288$ inches, $k = 10$, and E is taken as 30 000 000 pounds per square inch. The flexural unit-stress expressed in pounds per square inch is then

$$S_1 = 98\,016 \times 5.73/\left[157.4 - \frac{75\,630 \times 288 \times 288}{10 \times 30\,000\,000}\right] = 4115.$$

The direct compression due to the dead and live loads is $75\,630/11.76 = 6435$ pounds per square inch, and since the unit stress produced by the wind is greater than 30 percent of this, the wind stress must be considered in designing the section. The average allowable unit-stress for dead and live loads is

$$S' = 10\,000(2 \times 42\,380 + 2 \times 33\,250)/(2 \times 42\,380 + 33\,250)$$
$$= 12\,812,$$

expressed in pounds per square inch if the stress were direct. The specification column formula when wind is considered gives

$$P = 1.30(12\,812 - 1.28 \times 45 \times 288/3.66) = 10\,750 \text{ lb. per sq. in.}$$

The actual unit-stress due to dead and live loads is 6435 pounds per square inch; the unit-stress due to the direct wind compression (see Fig. 172) is $3765/(11.76 \times 2) = 160$ pounds per square inch; and that due to bending, as computed above, is 4115 pounds per square inch. The total actual unit-stress is then

$$S = 6435 + 160 + 4115 = 10\,700 \text{ pounds per square inch.}$$

Since this is less than the allowable unit-stress of 10 750 pounds per square inch, the channels need not be spaced farther apart than 5.97 inches back to back as stated above.

In case the entire bending moment due to wind had to be provided for, the total unit-stress would be $8230 + 320 + 6435 = 14\,975$ pounds per square inch, or $14\,975 - 10\,750 = 4225$ pounds per square inch more than that allowed. The channels would then have to be spaced a greater distance then 5.97 inches back to back. This distance is determined by trial, and usually two trials are sufficient to get within three or four percent, which is sufficiently close. A spacing of 7 inches back to back will make the post safe to resist the entire wind moment. Under the former conditions the channels must be at least 5.97, say 6 inches, back to back. On account of the width of the top chord

or end post, this distance may be increased in order to prevent excessive bending on the pins at the ends of the posts.

POSTS U_3L_3 AND U_4L_4. — Since the minimum section was used for U_2L_2, it must also be used for these posts. In case the stresses in the posts were such that the posts can be made of different sections, it is advisable to have their distances back to back equal to that required for the first post in order that the floor beams at L_2, L_3, and L_4 may have the same length.

END POSTS. — The section will consist of two channels with flanges turned out and a cover plate. Since the posts are spaced 6 inches back to back, the channels of the end post and chords should be at least this amount. In sections of this make-up the radius of gyration is approximately 0.4 the depth of the channel, and for equal radii of gyration about both rectangular axes the plate must be approximately $1\frac{1}{3}$ the width of the channel. Whenever possible the lightest weight (standard) channel of any size should be used, since it is easier to obtain from the mills. Also, the head of the largest eyebar, U_1L_2 must go inside of the chord at U_1, eccentricity being taken into account in case the pin is not put at the center of the channel webs. The eccentricity of the section is about $\frac{1}{6}$ the depth of the channels. The size of the eyebar head is dependent on the diameter of the pin and the excess of section area of the head over that of the body. The percentage of excess can be figured for any width of bar from the eyebar tables in the various manufacturers' handbooks. For a 5-inch bar in U_1L_2 it is 30 percent. The size of pin must be estimated. Table XI gives the sizes of pins which may be used for highway spans with a 16-foot roadway and a capacity of a 15-ton engine or 100 pounds per square foot of roadway. Sizes for other spans than those given may be interpolated.

TABLE XI. DIAMETER OF PINS.

Sizes modified where necessary by § 104 of the Specifications.

SPAN IN FEET.	WOODEN FLOOR.		CONCRETE FLOOR.	
	U_1 and Lower Chord.	Upper Chord.	U_1 and Lower Chord.	Upper Chord.
80	2¾ inches	2¼ inches	3 inches	2½ inches
100	3 inches	2½ inches	3¼ inches	2¾ inches
150	3¼ inches	2¾ inches	3½ inches	3 inches
200	4 inches	3 inches	4¼ inches	3¼ inches

The pin at U_1 is now estimated to be $3\frac{1}{4} + (4 - 3\frac{1}{4})(160 - 150)/(200 - 150) = 3.40$ inches, or say $3\frac{1}{2}$ inches. By § 104 of the specifications it must be $\frac{8}{10} \times 5 = 4$ inches. This will make the head of $U_1L_2 = 4 + 1.3 \times 5 = 10\frac{1}{2}$ inches. The pin being in the center of the channel web, the channel must be at least $(10\frac{1}{2}/2 + \frac{1}{2})\ 2 = 11\frac{1}{2}$ inches deep. Since there are no $11\frac{1}{2}$ inch channels, a 12-inch channel must be assumed. The width of the cover plate must be $1\frac{1}{3} \times 12 = 16$ inches.

The radius of gyration being approximately $0.4 \times 12 = 4.8$ inches, the allowable unit load (Cooper, § 45) is $10000 - 45 (29.85 \times 12)/4.8 = 7300$ pounds per square inch. The total equivalent live load stress is $105\,200/2 + 70\,800 = 133\,400$ pounds, which requires an area of $133\,400/6645 = 20.01$ square inches. Two 12-inch 25-pound channels have webs of allowable thickness and an area of $2 \times 7.35 = 14.70$ square inches. The required area of the 16-inch cover plate is $20.01 - 14.70 = 5.31$ square inches, and its thickness for this area is $5.31/16 = 0.33$ inches.

The channel flanges are 3.05 inches wide. Their outer edges should come slightly under the plate, say $\frac{1}{8}$ inch. This will require the distance back to back of channels to be not greater than $16 - (2 \times 3.65 + \frac{1}{8}) = 9.65$; and since the gage of the

channels is 1.75 inches, the distance between rivet lines must be not greater than $9.65 + 2 \times 1.75 = 13.15$ inches. If possible this should be a full quarter inch. It will therefore be taken as 13 inches, thus making the distance back to back of channels $13 - 2 \times 1\frac{3}{4} = 9\frac{1}{2}$ inches. According to Cooper, § 91, the least allowable thickness of this plate is $13/40 = 0.325$ inches. Accordingly the cover plates must be at least $\frac{3}{8}$ inch thick, and gives the following section area:

2 12-inch 25-pound channels	14.70 square inches
1 16 × $\frac{3}{8}$ inch cover plate	6.00 square inches
Total	20.70 square inches

This section must now be investigated to see that it is of sufficient area to allow for all the stresses which act upon it. The center of gravity is computed by taking the moments about an axis at the bottom flanges of the channels. Its distance from these flanges is $(6 \times 14.70 + 12\frac{3}{16} \times 6.00)/20.70 = 7.99$ inches. This shows the eccentricity to be $7.99 - 6 = 1.99$ inches. The moment of inertia about the axis 1–1 is best computed by arranging the computations in tabular form, thus:

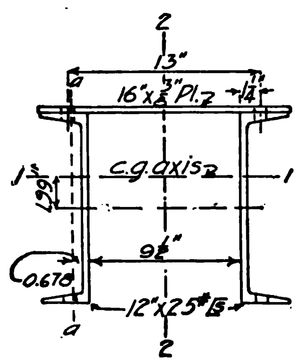


Fig. 173.

PIECE.	A	I'	h	Ah^2
2-Channels . .	14 70	288	1.99	58.20
1-Plate	6.00	——*	4.57	125.20
Sums . . .	20.70	288		183.40

* Neglected as it is very small.

whence $I = \Sigma(I' + Ah^2) = 471.4$ inches^4, and the radius of gyration of the section is

$$r = (471.40/20.70)^{\frac{1}{2}} = 4.77 \text{ inches.}$$

Lastly, by the column formula of the specifications,

$$P = 10\,000 - 540\,(29.85)/4.77 = 6625 \text{ pounds per square inch,}$$

which is the safe unit load for the assumed section. As the stress on the post, reduced to equivalent live load stress, is 133 400 pounds, the area required is $133\,400/6625 = 20.12$ square inches, which is practically the same as that assumed.

The direct stress due to wind is 18 720 pounds, which being less than 30 percent of the sum of the dead and live load stresses need not (Cooper, § 52) be considered.

The stress due to its own weight must be determined (Cooper, § 55). In order to compute this, an estimate of weight of the end post must be made, assuming the weight of the details to be 20 percent of the weight of the main section. The weight of the details will seldom vary much from this percentage in highway bridges such as are treated in these articles. The estimated weight is

2 Channels, 2 × 29.85 × 25	1493 pounds
1 Plate, 29.85 × 20.40	609 pounds
	2102 pounds
Details, 20 percent	420 pounds
Total	2522 pounds

The bending moment caused by this weight is $0.596 \times 2522 \times 29.85 \times 12/8 = 67\,200$ pound-inches, the term 0.596 being the sine of the angle of inclination the end post makes with the vertical. The unit-stress in the upper fiber due to this moment expressed in pounds per square inch is

$$S_1 = 67\,200 \times 4.385\Big/\left(471.4 - \frac{175\,800 \times 29.85^2 \times 12^2}{10 \times 30\,000\,000}\right) = 674,$$

which need not be considered, since it is less than 10 percent of the unit stress due to the dead and live loads, which is $175\,800/20.70 = 8490$ pounds per square inch.

The section must next be investigated in regard to its safety about the axis *2–2* in Fig. 173. The computations for the moment of inertia are arranged in tabular form:

PIECE.	A	I'	h	Ah^2
2 Channels . .	14.70	9.06	5.428	434.0
1 Plate. . . .	6.00	128.00	0	128.0
	20.70	137.06		562.0

from which $I = 137.06 + 562.0 = 699$ inches^4, and $r = (699/20.70)^{\frac{1}{2}}$ $= 580$ inches, which being greater than the radius of gyration referred to the axis *1–1* shows the section to be amply safe when dead and live loads only are considered.

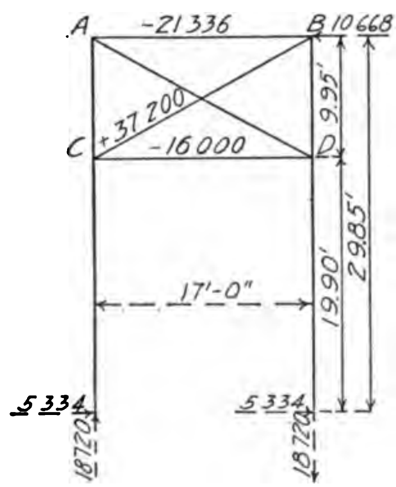


Fig. 174. — Wind Forces on End Post.

The section must also be examined to ascertain its safety when the wind stresses are considered. If the lower portal brace is at the same elevation as those of the intermediate cross-bracing, it will be $1.243 \times 8 = 9.95$ feet from the top. The wind forces acting on the portal and end post are as shown in Fig. 174. If the end post is fixed, the moment of one-half the stress times the distance back to back of channels must be greater than the moment of the wind reaction at the foot times one-half the distance from D, Fig. 174, to the foot of the end post. The former is

$$175\,800 \times 9.50/2 = 836\,000 \text{ pound-inches,}$$

and the latter is

$$10\,668 \times 19.90 \times 12/2 = 1\,236\,500 \text{ pound-inches.}$$

This shows the post to be free or at least partially so. According to the methods of mechanics, if the post was absolutely fixed, the point of inflection would be at one-half the distance from the foot of the end post to the lower portal strut; and if it is partially fixed, the point will be at some intermediate point. The moment 836 000 shows this point to be $836\,000/5334 = 156.5$ inches from the foot of the portal. The maximum bending moment occurs at the end of the lower portal strut and is

$$5334\,(19.9 \times 12 - 156.5) = 438\,000 \text{ pound-inches.}$$

The unit-stress expressed in pounds per square inch is

$$S_1 = (438\,000 \times 16/2)/\left[699 - \frac{(175\,800 + 18\,220)(29.85 \times 12)^2}{10 \times 30\,000\,000}\right]$$
$$= 5620.$$

The average unit-stress produced by both dead and live loads is $175\,800/20.70 = 8470$ pounds, and since the wind produces a stress greater than 30 percent of this, the wind must be considered. According to equation (1) in Art. 116 the coefficient by which the allowable unit-stress for live load must be multiplied is $(2 \times 70\,800 + 2 \times 105\,000)/(2 \times 70\,800 + 105\,000) = 1.4255$. The allowable unit load is $P = 10\,000 - 540 \times 29.85/5.80 = 7230$ pounds per square inch, and, considering wind, the allowable unit-stress is $7230 \times 1.4255 = 10\,300$ pounds per square inch. The actual unit-stress is $(175\,800 + 18\,720)/20.70 + 5620 = 14\,747$ pounds per square inch, which shows the section to be deficient and a revision necessary.

The section can be increased either by using heavier 12-inch channels or by using the same channels and placing them farther apart, a wider and thicker plate being used; but it is more economical to use wider channels, the plate being widened accordingly. If 15-inch 35-pound channels are used, the plate must

be about $1\frac{1}{3} \times 15 = 20$ inches wide and $\frac{7}{16}$ inches thick, the channels being placed 13 inches back to back. Such a section was tried and found to be a little too large. A section consisting of two 15-inch 33-pound channels, 13 inches back to back, with a cover plate $19 \times \frac{7}{16}$ inches gives an area of 28.11 square inches. The moments of inertia and radii of gyration were found to be

Axis of Reference	I	r
Perpendicular to channel web	975	5.89
Perpendicular to cover plate	1385	6.51

The allowable unit-stress, including wind, is $1.4255 \times (10\,000 - 540 \times 29.85/6.51) = 10\,700$ pounds per square inch; the actual unit-stress, with wind considered, is $(175\,800 + 18\,720)/28.11 = 6917$ pounds per square inch; and the flexural unit-stress caused by the wind is

$$S_1 = (438\,000 \times 19/2)/\left[1385 - \frac{(175\,800 + 18\,720)(29.85 \times 12)^2}{10 \times 30\,000\,000}\right]$$
$$= 3175,$$

making the total unit-stress $6917 + 3175 = 10\,095$ pounds per square inch, which, being slightly less than the allowable value of 10 700, shows that the section is correct.

The section must be examined still further for eccentricity of pins with respect to the axis perpendicular to the channel webs. If the pins are not on the center of gravity axis, stresses due to eccentricity will occur. Since no bending due to wind occurs in the top chord, it is possible to use 12-inch channels with the pin at the center of the web. For this reason it is advisable to assume that the center line of pins in the end post be 6 inches from the cover plate. This makes the eccentricity $6 - (15/2 - 2.28) = 0.78$ inch, the term 2.28 being the distance from the center of gravity of the section to the center line of the webs of the channels.

Referred to the axis perpendicular to the channel webs, the allowable unit-stress is 1.4255 (10 000 − 540 × 29.85/5.89) = 7260 pounds per square inch on the basis of live load equivalent. The actual average unit-stress on the same basis is 123 400/28.11 = 4390 pounds per square inch, while the bending moment due pin eccentricity is 123 400 × 0.78 = 96 300 pound-inches. The stress on the lower fiber due to this, expressed in pounds per square inch, is

$$S_1 = 96\,300 \times 8.28 \Big/ \left[975 - \frac{123\,400(29.85 \times 12)^2}{10 \times 30\,000\,000}\right] = 853.$$

The total actual stress is 4390 + 853 = 5243 pounds per square inch, which being less than 7260, the allowable unit-stress, the section is shown to be safe. It is much safer about the horizontal axis than about the vertical axis.

ART. 118. TOP CHORD SECTIONS.

In highway bridges such as are treated in this chapter it is more economical to determine the section for that chord member having the greatest stress, and to make the top chord of the same section throughout.

The member U_4U_4 has a dead load stress of 157 200 pounds and a live load stress of 105 600 pounds. The total wind stress being 5580 pounds need not be considered (§ 52, Cooper). The total equivalent live load stress is 157 200/2 + 105 600 = 184 200 pounds. If 12-inch channels with a 19-inch cover plate are assumed to be sufficient, the approximate allowable unit load is 12 000 − 55 × 18.78 × 12/4.8 = 9555 pounds per square inch, and the approximate required area is 184 200/9555 = 19.30 square inches. The plate must be at least 15¾/40 = 0.394, or $\frac{7}{16}$ inch thick (§ 91, Cooper), giving a section area of 8.31 square inches. This makes the required approximate area of one channel (19.30 − 8.31)/2 = 5.50 square inches, which requires a

12-inch 25-pound channel at least, provided there are no other stresses but those due to the dead and live loads. There will be considerable additional stress due to the eccentricity of the pins, which must be in the center of the web, and therefore over an inch from the center of gravity axis. Two 12-inch 30-pound channels are not sufficient. Two 12-inch 35-pound channels might be used. Two 15-inch 33-pound channels are better. Although their area is less than the 35-pound channel, they are deeper, and the actual total unit-stresses will be smaller. The same section as was used in the end post will be assumed as sufficient and will be investigated.

The actual unit-stress due to equivalent live load is $(157\,200/2 + 105\,600)/28.11 = 6550$ pounds per square inch. The allowable unit-stress on the basis of live load is $12\,000 - 660 \times 17.78/5.89 = 10010$ pounds per square inch. The eccentricity of the pins being the same as in the end post, the moment caused by it is $184\,200 \times 0.78 = 143\,500$ pound-inches, and the unit-stress in the lower fiber due to this, expressed in pounds per square inch, is

$$S_1 = 143\,500 \times 8.28 \Big/ \left[975 - \frac{184\,200\,(17.78 \times 12)^2}{10 \times 30\,000\,000} \right] = 1255.$$

The maximum compressive stress is in the extreme lower fiber, and is $6550 + 1255 = 7805$ pounds per square inch. Although this shows that there is an excess of material in the section, it will be used, since investigation of other sections show this to be the most economical.

ART. 119. LATERAL AND CROSS BRACING.

With a concrete floor, it is not essential that stiff lateral diagonal members be used; with plank floors, it is. In this case round rods upset at the ends and threaded will be used for the top and bottom lateral diagonals, and square loop bars will be used for the diagonals of the transverse and portal bracing.

Whenever loop bars are used they should be square or rectangular in section, never round, in order to give a good bearing on the pin. One diagonal in each of the intermediate and portal bracings should be made adjustable. The allowable unit-stress (§ 45, Cooper) is 18 000 pounds per square inch. The computations are best arranged as follows, reference being made to Figs. 169, 172, and 174, and to § 97 for the specifications.

REFERENCES.	MAXIMUM STRESS. POUNDS.	AREA REQUIRED. SQUARE INCHES.	SECTION USED.
Top Laterals			
1st panel	+ 11 520	0.64	1 rod 1″ diameter
2d panel	+ 7 680	0.43	1 rod 1″ diameter
3d panel	+ 3 840	0.21	1 rod 1″ diameter
Center panel	+ 0	0	1 rod 1″ diameter
Bottom Laterals			
1st panel	+ 45 900	2.50	1 rod $1\frac{5}{8}$″ square
2d panel	+ 23 470	1.31	1 rod $1\frac{3}{16}$″ square
3d panel	+ 16 640	0.93	1 rod 1″ square
4th panel	+ 10 240	0.57	1 rod $\frac{7}{8}$″ square
Center panel	+ 2 820	0.16	1 rod $\frac{7}{8}$″ square
Intermediate			
Cross-bracing	+ 8 850	0.49	1 loop bar $\frac{7}{8}$″ × $\frac{7}{8}$″
Portal			
Diagonals	+ 37 200	2.00	2 loop bars $1\frac{1}{16}$″ × $1\frac{1}{16}$″

The best section for the struts of the intermediate transverse bracing is one consisting of two channels latticed. This will allow for better connections than any other section (see Figs. 158 and 159). Here the length is taken as the clear width of roadway, 16 feet. By § 48 of the specifications, the radius of gyration must not be less than $16 \times 12/120 = 1.6$ inches. Two 5-inch $6\frac{1}{2}$ pound channels satisfy this condition, their radius of gyration being 1.95. They must not be spaced less than $2\frac{7}{8}$ inches back to back, flanges out. The allowable unit load

(§ 48, Cooper) is $13\,000 - 720 \times 16/1.95 = 7090$ pounds per square inch. The required area (see Fig. 174), is $5334/7090 = 0.752$ square inches, and the given area is $2 \times 1.95 = 3.90$ square inches. These channels are of sufficient area and will be used.

The total compression in the portal struts is 21 336 pounds and the required area, assuming the same section as used in the intermediate struts, is $21\,336/7090 = 3.01$ square inches. This shows the section to be sufficient, and it will be used.

ART. 120. DETAILING.

Under this head come the determination of the sizes of pins, lacing, and batten or stay plates; the number of rivets; the thickness of pin plates; the number, diameter, and length of rollers; and of the sizes of bearing and bed plates. Since this ground is covered in Chap. IX, the reader is referred to that for his guidance, careful attention being paid to the clauses of the specifications which relate to the several items of design. The most important clauses are the following: For pins, §§ 54, 102, 103, and 104; for lacing and batten plates, § 110; for rivets, §§ 53, 66, 75, 90, 109, and 111; for pin plates, §§ 53 and 112; for rollers, §§ 127 and 128; and for bearing plates, §§ 126, 129, and 132.

The sections should be marked on the stress sheet, and also sketches showing any particular detail designed by the engineer. It is not advisable to specify any particular detail unless the designer is familiar with shop practice. Different shops may have different details, and each shop's own detail may be, and usually is, more economical for it to make. It is better to submit the stress sheet, and after the contract is let, to criticise the shop drawings which will be furnished for approval.

ART. 121. HIGHWAY BRIDGES — REFERENCES.

The student should consult the following selected references to engineering periodicals to acquire a knowledge of other details of highway bridge trusses and floor systems than those described and illustrated in this chapter.

HIGHWAY PIN BRIDGES.

Highway Bridge of 319-ft. Span, Livermore Falls, Me. Eng. News, v. 37, p. 191, Mar. 25, 1897.

Highway Bridge of 406-ft. Span, Hamilton, O. Eng. News, v. 45, p. 370, May 23, 1901.

South Tenth Street Bridge, Pittsburgh. Eng. Rec., v. 50, p. 682, Dec. 10, 1904.

Mercantile Bridge Across the Monongahela River. Eng. Rec., v. 58, p. 686, Dec. 19, 1908.

Webster-Donora Bridge. Eng. Rec., v. 59, p. 753, June 12, 1909.

Design of the Broadway or Sparkman Street Bridge, Nashville, Tenn. By H. M. JONES. Eng. News, v. 62, p. 570, Nov. 25, 1909.

Hudson River Bridge at Waterford, N. Y. By H. N. PECK. Eng. News, v. 64, p. 33, July 14, 1910.

HIGHWAY RIVETED BRIDGES.

Street Bridges over Railroad Tracks in Buffalo. Eng. Rec., v. 39, p. 539, May 13, 1899.

Erection of the Portage du Fort Bridge. Eng. Rec., v. 45, p. 26, Jan. 11, 1902.

Rutland Canadian Railway and its Structures; Overhead Steel Highway Bridge. By JOHN W. BURKE. Eng. News, v. 49, p. 46, Jan. 15, 1903.

Steel Truss Highway Bridge with Concrete Floor. Eng. Rec., v. 47, p. 423, Apr. 25, 1903.

Springville Bridge. Eng. Rec., v. 47, p. 634, June 13, 1903.

Greenfield Street Railway Bridge. Eng. Rec., v. 49, p. 462, Apr. 9, 1904.

Merrimac River Bridge at Newburyport. Eng. Rec., v. 50, p. 218, Aug. 20, 1904.

Highway Bridge over the Wabash River at Terre Haute. Eng. Rec., v. 51, p. 672, June 17, 1905; R. R. Gaz., v. 39, p. 20, July 7, 1905. By M. A. HOWE. Eng. News, v. 55, p. 273, Mar. 8, 1906.

Jefferson Street Bridge, Newark, N. J. Eng. Rec., v. 55, p. 181, Feb. 16, 1907.

Combined Concrete and Steel Girder Bridge, Monroe Street, Brookland, D. C. By W. J. DOUGLAS. Eng. News, v. 60, p. 464, Oct. 29, 1908.

Carolina, Clinchfield, and Ohio Railway; Details of Overhead Highway Bridge. Eng. News, v. 61, p. 53, Jan. 21, 1909.

Communipaw Avenue Bridge, Central Railroad of New Jersey. Eng. Rec., v. 59, p. 187, Feb. 13, 1909.

Steel Highway Bridge with Concrete Stringers and Floor Slabs. Eng. Rec., v. 59, p. 334, Mar. 20, 1909.

Recent Short-Span Highway Bridges. Eng. Rec., v. 59, p. 351, Mar. 27, 1909.

Sixth Street Viaduct, Kansas City. By E. E. HOWARD. Trans. Am. Soc. C. E., v. 65, p. 42, Dec., 1909; R. R. Gaz., v. 42, p. 521, Apr. 12, 1907; Eng. News., v. 58, p. 323, Sept. 26, 1907.

Bailey Avenue Bridge, Buffalo. Eng. Rec., v. 61, p. 612, May 7, 1910.

CHAPTER XI.

RAILROAD RIVETED BRIDGES.

ART. 122. FORMS OF TRUSSES.

As stated in Art. 70, the lower limit of span for railroad riveted trusses ranges from 100 to 110 feet, and the upper limit from 150 to 250 feet, according to many specifications and standards. In practice, however, the tendency is to extend the upper limit of span considerably. In 1910 and 1911 at least seven railroad bridges were under construction in the United States containing riveted truss spans over 250 feet, and three of them contain spans over 350 feet, the largest being 425 feet 6½ inches. In Canada the longest riveted span is 412 feet 8 inches. A number of bridges have been built in which the truss joints are partly pin-connected and partly riveted (see Part I, Fig. 71*a*). This arrangement will probably be adopted often for large spans in connection with stiff lower chords in order to employ the cantilever method of erection over navigable streams and which it may not be so economical to rivet throughout. The general tendency is to restrict more and more the use of purely pin-connected truss bridges.

The types of trusses in most general use for riveted bridges are the Warren with sub-verticals, the Pratt, the Parker, and the Baltimore. The order here given indicates in general their order of application as the span increases, except that the Baltimore truss may be used for shorter spans than the Pratt or Parker trusses when the use of solid floors requires very short

Fig. 175.—Raft Span of Chicago & Northwestern Railroad Bridge over Mississippi River at Clinton, Ia. Erected in 1908.

panels. The double-intersection Warren truss is also used to a

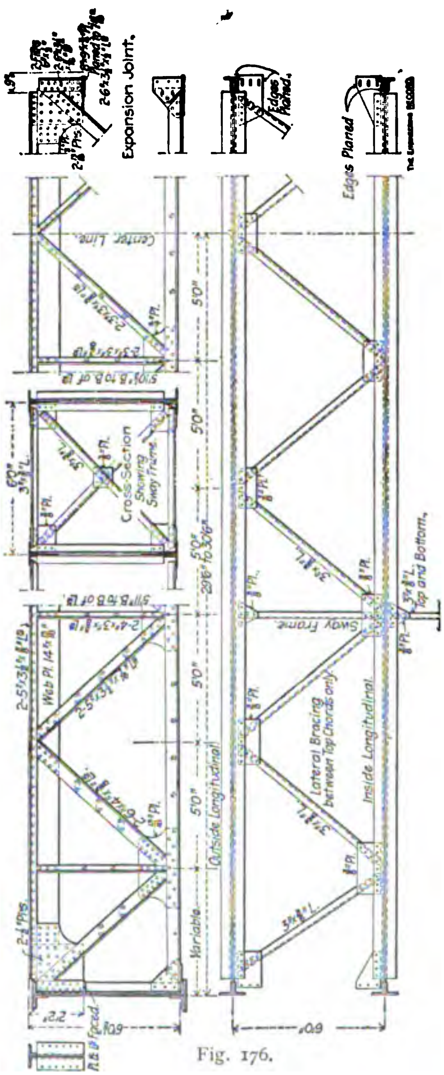


Fig. 176.

limited extent, sub-verticals being added when necessary to secure a shallow floor. The quadruple-intersection Warren truss is used occasionally, and since 1900 riveted Whipple trusses have been introduced for several bridges in the West. The use of more than a single system of web members is not regarded with favor in the best practice, because the stresses are not statically determinate and the metal is distributed in numerous members with small cross-sections. In some instances the stresses in double-intersection Warren trusses are made indeterminate to a still higher degree by the insertion of long verticals connecting the two systems. This is contrary to the line of progress described in Chap. I.

In 1911 the general practice is to eliminate the pony riveted truss, and to pass directly from the through plate girder to the through riveted

truss, with the same lateral, portal, and sway bracing as those employed for pin bridges (see Chap. VIII).

On elevated railroads, girders whose spans range from 40 to 65 feet are often required to be built with open webs in order to admit more light beneath the structure than the solid webs of plate girders. This requirement applies to most locations except those where the elevated structures occupy the middle of a very wide street. Fig. 176 shows the plan, elevation, and cross-section of a half span of the Boston Elevated Railroad. The deck trusses, whose depth is very nearly six feet, are of the Warren type with sub-verticals. The upper chord is subject to combined compression and flexure, and is composed of a pair of angles and a web plate, while the lower chord and all of the web members consist only of pairs of angles, separated by washers whose thickness equals that of the connecting plates. The lateral and sway bracing is the same as that of a deck plate-girder bridge.

ART. 123. COMPOSITION OF MEMBERS.

An examination of numerous general drawings and standard plans of railroad riveted truss bridges for spans not exceeding 200 feet is the basis of the following general statements in regard to the composition of truss members in the United States. Some of the designs were made by consulting bridge engineers, others by the bridge departments of railroads, and the rest by the engineering departments of bridge works.

The sub-verticals of the Warren truss consist most frequently of two pairs of angles laced together (see Fig. 86 in Chap. VIII), while less frequently the lacing is replaced by a solid web plate. In the longer spans or in double-track bridges they are made up of two channels laced together (Figs. 79 and 80). When sub-verticals extend from panel points near the floor to the other

chord and hence serve only to hold that chord in line, they are sometimes composed of two single angles laced together.

The diagonals of the Warren truss are usually composed either of two pairs of angles united by a web plate, or of two channels laced together. In some cases the former section is increased by adding two side plates, while in others it is diminished by omitting the web plate. As a rule, no distinction is made between the composition of the diagonal ties and struts. In the longer spans or in double-track bridges, built-up channels are substituted for the channel shapes when the latter fail to provide sufficient section area (Figs. 83 and 84).

The suspenders or hip verticals of the Pratt truss consist most frequently of two pairs of angles connected by a web plate, while less frequently the web plate is replaced by single lacing. Occasionally they are made up of two channels which are laced together on both sides of the member.

In the main diagonals of the Pratt truss, laced channels are used much more frequently than the angles and web plate. The former section is sometimes increased by adding a plate to the web of each channel, or by substituting a built channel, but the latter section is rarely modified by either adding or omitting any plate or shape. When the first diagonal is subject only to tension, and the second has to resist both tension and compression, the former may have two pairs of angles laced while the latter has two channels laced together. In the middle panel both counter diagonals consist almost exclusively of two pairs of angles laced, and the same composition is used for counters which are inserted in adjacent panels. The tendency is to use but few counters in comparison with former practice. The intermediate posts usually have the same composition as the main diagonals.

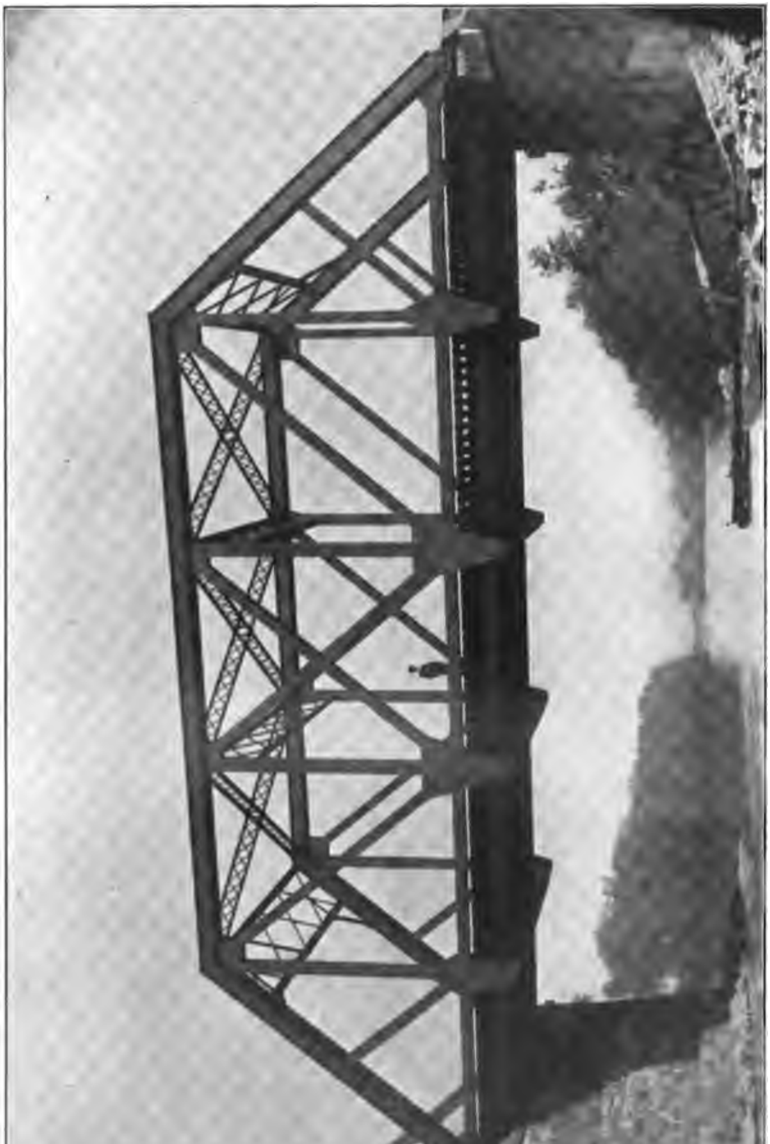


Fig. 177.—Central Railroad of New Jersey Bridge over Lehigh River at Allentown, Pa. Built in 1903.

For the lower chord of both Warren and Pratt trusses, with the shortest spans, two pairs of angles connected by horizontal lacing are used, this section being increased toward the middle of the span by adding side plates (see Figs. 87 and 88). For larger spans and heavier loading two channels laced on both top and bottom are used, the section being increased by additional side plates. Built channels are employed next, the section area of each side of the member being increased by an additional plate on the back of the angles, or by a filler plate between the vertical legs of the angles, and a side plate on the face of the angles, as indicated in Fig. 94. For single-track spans below 150 feet, the channel shapes are much more frequently used than either of the other two compositions mentioned.

The corresponding upper chord sections consist respectively of two channels laced on both top and bottom; two channels with a cover plate on top and lacing on the bottom; and two built-up channels with cover plate and lacing. The built-up section is used far more extensively than any other form of upper chord. Occasionally its cover plate is replaced by lacing. In some cases the center of gravity is brought approximately to mid-depth either by the addition of flats as in Figs. 95 and 96, or by using angles of different sizes as in Fig. 97. In all cases where necessary the section is increased by additional filler and side or web plates.

The composition of the end post, as a rule, is identical with that of the upper chord in the same bridge, with the exception, however, that the cover plate is very rarely omitted on account of the lateral flexure to be resisted by the end post.

In Canada it is regarded as good practice to omit all lacing on the web members excepting the end post. The smaller sections consist of two pairs of bulb angles connected by a web plate.

Fig. 178. — Canadian Pacific Railway Bridge over Kootenay River, Wardner, B. C. Built in 1909. All web members except the collision struts and end posts are constructed without lacing.

Fig. 179.—Grand Trunk Railway Bridge over St. Lawrence River at Coteau, Que. Completed in 1910.
All web members except the end posts are constructed without lacing.

Fig. 180. — End View of a Portion of the Bridge at Coteau, Que.

The Grand Trunk Railway bridge over the St. Lawrence river at Coteau, Quebec, Canada, consists of the following spans and lengths between centers of end bearings, beginning at the north end: One span of 135 feet; a swing span of 350 feet 9 inches; two spans of 171 feet to Giroux island; ten spans of 213 feet to Round island; and four spans of 219 feet to Clarke's island. The three shorter spans have Pratt trusses, while the remaining fourteen fixed spans have Parker trusses. The original bridge was completed in 1890, and in 1910 the present superstructure replaced the old riveted truss spans, which were too light for the requirements of modern traffic.

Those next in size are increased by the addition of two side plates slightly narrower than the width out to out of bulbs. Ordinary angles are substituted for the bulb angles in the suspenders of Pratt and Parker trusses and in both counter diagonals of the middle panel.

This composition is simple in construction, as only two rows of rivets are required for the smaller sections, and in any case the number of pieces to be assembled is small. The bulbs on the angles increase the strength and stiffness of the member materially, and experience proves their increased resistance to deformation during construction, shipment, and erection. The extensive use of bulb angles in shipbuilding for many years demonstrates their value in structural work. In the riveted truss designs of the Grand Trunk Pacific Railway the compositions of web members noted in the preceding paragraph were adopted as standard. The improved appearance of trusses containing them is noticed at a considerable distance, whether they are approached from the side or from the end of the bridge.

An important change has also been made in Canada in the composition of the end post. Instead of connecting the built-up channels of the sides by a cover plate on top and double lacing below, they are united by a central longitudinal diaphragm consisting of a solid web plate and two pairs of angles, in addition to lacing on both top and bottom surfaces of the end post. This section has been adopted in the standard plans of the Great Northern Railway.

ART. 124. WIDTH OF TRUSS MEMBERS.

In riveted truss bridges the widths of the web and chord members are more closely related to each other than in pin truss bridges, since all the members meeting at any panel point are usually connected by means of two plates. This relation of

width measured in a direction perpendicular to the plane of the truss depends also upon the direction in which the channel flanges or angles are turned.

In vertical posts and suspenders the flanges of channels are nearly always turned in. In diagonals they are either turned in or out, but the practice of turning them in is steadily increasing. In the lower chord the prevailing practice is to turn the channel flanges in since this materially simplifies the floor-beam connections, but when the chord is built up with angles and plates the practice of turning the flanges in or out is equally divided. In some cases the upper flanges are turned in while the lower ones are turned out. An additional practical reason for turning in the flanges on the lower chord is to avoid collecting cinders and moisture on the horizontal leg of the lower inner flange angle. In the upper chord and end post, however, the flanges are nearly always turned out.

The illustrations in this chapter show some of the arrangements of members and connecting plates at panel points. It is usually desirable to avoid the use of filler plates between them as far as possible. Sometimes the connecting plates at the hip and end joints of the truss are made slightly thicker than those at the other joints.

An examination of over sixty selected standard and other plans of railroad riveted trusses, prepared by bridge engineers in various parts of the country since 1900, reveals wide variations in the widths of the truss members for the same spans and loading. For example, in single-track bridges with a span of 150 feet, the distance back to back of the upper chord flange angles varies from 12¾ to 18 inches; and for a span of 125 feet, from 11 to 16¾ inches. Again, a spacing of from 16 to 16½ inches is used for single-track spans ranging from 105.5 to 197.5 feet.

Fig. 181.—Central Railroad of New Jersey Bridge over Lehigh River at Bethlehem, Pa. Built in 1908.

While it is not possible to state precisely what the average practice is, an approximate guide for the student may be given, based on the above examination.

For single-track bridge trusses the distance, in inches, back to back of upper chord flange angles may be taken equal to $8 + 4.75\,l$, in which l is the span of the truss in feet. In practice, the spacing varies approximately $\pm 2\frac{3}{4}$ inches from the average given by this formula. For double-track bridges the distance may be taken as $8 + 7.5\,l$, but the data upon which this is based are meager as compared with those for single-track bridges.

When the sides of the upper chord consist of channel shapes, the distance given is that back to back of the channels, but when they are built up with web plates and angles, the distance given is measured between the backs of the angles. In the latter case, therefore, the thickness of the web plates must be included as well as that of the connecting plates in computing the corresponding widths of the web members. The thickness of connecting plates is discussed in the next article.

The relation between the width of the end post and that of the lower chord depends upon the arrangement of these members at the end panel point of the truss. In practice, two plans are most frequently employed. In the first plan the end post is extended downward past the lower chord, while the latter is cut off so as to be riveted only to the main connecting plates (see Fig. 177 and Plate VI); while in the second plan the lower chord is extended past the end post and the latter is cut off just above the chord (see Part I, Fig. 29*b*). The first plan is used more frequently than the second, but the difference does not exceed probably twenty percent.

Another important feature of the end joint of a riveted truss is the location of the pin connecting it to the end bearing. The

pin is either placed at the intersection of the neutral planes of the end post and lower chord (Plate VI), or at some distance vertically below that position (Fig. 177). In the latter case the pin is generally placed some distance below the bottom of the lower chord. Between these two positions, practice is about equally divided. Furthermore, each of these positions of the pin is combined to about the same extent with each of the two plans mentioned in the preceding paragraph.

Occasionally, when the flanges of the lower chord are turned in, both the end post and lower chord are extended past each other so that a considerable number of rivets pass through both members as well as the connecting plate. In this case the pin is placed below the lower chord. Other minor modifications are made, one example of which is shown in Fig. 175, and another on Plate VII.

ART. 125. RIVETED CONNECTIONS.

Joints and connections in riveted work, whether in tension or compression, are designed to develop the full strength of the members, proper allowance being made for field riveting. The connecting plates must have a thickness porportioned to the amount of stress to be transferred, and must properly distribute the components of the web stresses to the plates and shapes which compose the chords. Sometimes the main connecting plates at a panel point also perform the additional function of splice plates for the lower chord.

When but few lines of rivets are used in any connection and the lines are long, the elongation of the member within the limits of the connection makes a very unequal distribution of the stress to the rivets. This consideration will often determine whether to employ angles with legs wide enough to permit two rows of rivets or one. In the connections of web members

composed of channel shapes or of built-up channels, the length of the group of rivets is frequently shortened for the same reason by the addition of angle clips or lugs riveted to the channel flanges, thus increasing the number of rows of rivets attaching the member to the connecting plate. Sometimes the thickness of plate is made to depend upon the number of rivets in the connections, as, for instance, in the 1901 specifications of the Baltimore and Ohio Railroad, which limit the number of rivets in any connection of a $\frac{3}{8}$-inch plate to ten.

An examination of the same plans mentioned in the preceding article reveals a remarkable variation in practice regarding the thickness of the large plates which connect the truss members at the panel points. For example, $\frac{1}{2}$-inch plates are used in the trusses of single-track bridges with spans from 100 to 230 feet, and in double-track bridges with spans of 100 to 155 feet; while $\frac{5}{8}$-inch plates are used in single-track bridges with spans of 105 to 275 feet, and in double-track bridges with spans of 100 to 180 feet. The average spans for the four cases just mentioned are 148, 130, 193, and 130 feet respectively. For $\frac{7}{16}$-inch plates the spans of single-track bridges vary from 110 to 150 feet, the average being 132 feet.

No rational method for the design of connecting plates has yet been developed. As a pre-requisite for such design observations are needed regarding the distribution of stresses in very large plates by precise measurements of their deformation. Apparently no published results of inspections of trusses under traffic are available which might aid in the solution of the problem. The following approximate method to determine the thickness of connecting plates may be adopted as a guide or check. Let it be assumed that the stress for each side of any web member is transmitted into the corresponding connecting plate between two lines starting from the intersections of the

edges of the member and plate and diverging at angles not exceeding 30 degrees with the axis of the member. Let it be assumed, further, that the stress in the plate is uniformly distributed in a cross-section, perpendicular to the axis of the member, taken slightly beyond its last row of connecting rivets, and which is limited in length by the diverging lines. In some cases its length will be limited still further by an edge of the plate itself, and then it may also become necessary to consider the effect of eccentricity. The greatest thickness thus required by the connection of the plate with any member will then indicate that to be adopted.

For long spans and heavy loading the connecting plates are built up of two or more thicknesses of plates having different surface areas. In the riveted trusses of the Terminal Railroad bridge over the Missouri river at Kansas City, with spans of 423 to 425.5 feet, three thicknesses are used, the largest single plate being $131\frac{1}{4}$ inches wide, $\frac{15}{16}$ inch thick, and 18 feet 10 inches long. In this case the web plates are spliced into the connecting plates. More than two connecting plates at any panel point, one on each side of the truss, are rarely required for spans below 250 or 300 feet.

Tests of riveted joints show that the working strength of a joint is governed not only by the number of rivets used, but also by their arrangement and that of the component pieces of the joint. When the stress is transferred indirectly through filler plates, additional rivets are required to connect the fillers to the main plates or shapes in order to develop the full strength of a similar joint without fillers. The following selected references relate mainly to tests on the strength of riveted joints.

Net Section in Riveted Work. By THEODORE COOPER. R. R. Gaz., v. 22, p. 583, Aug. 22, 1890.

Experiments on Iron and Steel Joints, Riveted on an Angle. By B. B. FLINT. Trans. Am. Soc. C. E., v. 27, p. 406, Oct., 1892.

Watertown Tests on Riveted Joints. Tests of Metals, etc., 1895, pp. 281–289; 1896, pp. 245–339.

Tests of Riveted Joints. Proc. Am. Ry. Eng. & M. W. Assoc., 1905, v. 6, pp. 272–446.

Distribution of Stress in Riveted Connections. By C. R. YOUNG. Trans. Can. Soc. C. E., 1906, v. 20, pp. 257–287.

Some Tests Bearing on the Design of Tension Members. By EDWARD GODFREY. Eng. News, v. 55, p. 488, May 3, 1906.

The Strength of Riveted Joints. By THEODORE COOPER. Eng. News, v. 55, p. 520, May 10, 1906.

Tension Tests of Steel Angles with Various Types of End Connection. By F. P. McKIBBEN. Eng. News, v. 58, p. 190, Aug. 22, 1907; v. 56, p. 14, July 5, 1906; Eng. Rec., v. 54, p. 148, Aug. 11, 1906.

Tables for Computing Eccentric Riveted Joints. By M. R. HULL. Eng. News, v. 62, p. 204, Aug. 19, 1909.

Nickel Steel Riveted Joints. Editorial. Eng. Rec., v. 61, p. 569, Apr. 30, 1910.

Tests of Nickel Steel Riveted Joints. Eng. Rec., v. 62, p. 200, Aug. 20, 1910; editorial, p. 197.

Tests of Nickel Steel Riveted Joints. By ARTHUR N. TALBOT and HERBERT F. MOORE. Bulletin No. 49, University of Illinois Engineering Experiment Station, 1911.

ART. 126. DETAILS OF A PRATT TRUSS.

Plate VI contains parts of a general drawing of a riveted Pratt truss whose span is 170 feet. It was taken from one

of a series of standard plans of riveted bridges having a considerable range of span, and which were designed for class W of WADDELL's compromise standard live loads (Art. 32). All the material is medium steel except the rivets and anchor bolts, which are of soft steel. The trusses are spaced 17 feet apart between centers, while the stringers are spaced 7 feet apart.

The stringers (not shown on Plate VI) have web plates $39\frac{3}{4}'' \times \frac{3}{8}''$, and flanges of two angles $6'' \times 4'' \times \frac{1}{2}''$, the long legs being horizontal. There are seven intermediate pairs of stiffeners crimped over the flange angles. The end connecting angles have fillers under them twice as wide as the angles. The lateral system of the stringers in each panel consists of four diagonals, each composed of one $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle. The stringers are connected by means of two short angles to the lower laterals of the bridge in a very effective manner in accordance with the specification given in Art. 94. The connection of one of the transverse braces between the lower flanges of the stringers is shown on the plate.

The floor beams have intermediate stiffeners. Their connection to the posts and suspender is very simple, since these members have the same width as the stiff lower chord. The web of the end floor beam is reënforced at each end by a plate 25 inches wide inside of the end connections. A diaphragm like that in the verticals is placed between the large connecting plates at the panel point L_0.

The posts and suspender have sections like those for pin-connected trusses. The diagonal U_1L_2 consists of two plates and four angles united by a single line of lacing, while the diagonal U_2L_3 has two channels with the flanges turned inward and united by two lines of lacing. In the middle panel there are two stiff diagonals, each composed of two pairs of angles

connected by a single line of lacing. Both diagonals are cut at their intersection and riveted to a pair of connecting plates.

The upper chord and end post consist of a cover plate and two channels connected below by tie plates and single lacing. The splices of the upper chord are similar to those of a pin-connected truss. From L_0 to L_2 the lower chord section is composed of two web plates and four angles united by a series of narrow tie or batten plates, as shown on the drawing. From L_2 to L_3 two plates of the same section are added and the angles increased to $4'' \times 3'' \times \frac{7}{16}''$, while in the middle panel the chord consists of two plates $12'' \times \frac{9}{16}''$, two plates $12'' \times \frac{5}{8}''$, and four angles $4'' \times 3'' \times \frac{7}{16}''$. The entire chord is spliced on the left of L_2, the composition of the splice being given on the plate. There is a similar splice also at the left of L_3.

The upper laterals are made up of two angles $4'' \times 3'' \times \frac{3}{8}''$, laced together so as to form a stiff member as deep as the chord, and attached by connecting plates to the top and bottom of the lateral struts as well as to the chords. Each of the lower laterals consists of two angles $4'' \times 3' \times \frac{3}{8}''$, placed with the $4''$ leg vertically and riveted together every foot. The splice at the intersection of the laterals has two angles of the same size in addition to the $12'' \times \frac{3}{8}''$ plate in order to give stiffness as well as strength to the splice. The end connecting plates are riveted to the bottom flanges of the floor beams and to the shelf angles attached to the side of the chords.

The portal bracing consists of two small trusses, one of them connected to the upper and the other to the lower side of the end posts. All the corresponding members of these trusses are laced together in pairs, thus making a portal of considerable stiffness in all directions. The construction of the intermediate sway bracing is fully shown on the drawing.

The connecting plates at the different panel points are all $\frac{1}{2}$ inch in thickness. They are riveted to the inside of the upper chord and end posts, and to the outside of all the other members. The reaction of the panel point L_0 is transferred from the end post to the pedestal by a 6-inch pin, the necessary bearing of the end post being secured by means of $\frac{5}{8}$-inch pin plates in addition to the large connecting plates. On account of the eccentric location of the pin, two angles are inserted to reduce the effect of this eccentricity on the pin plates and to aid in transferring its share of the stress into the cover plate.

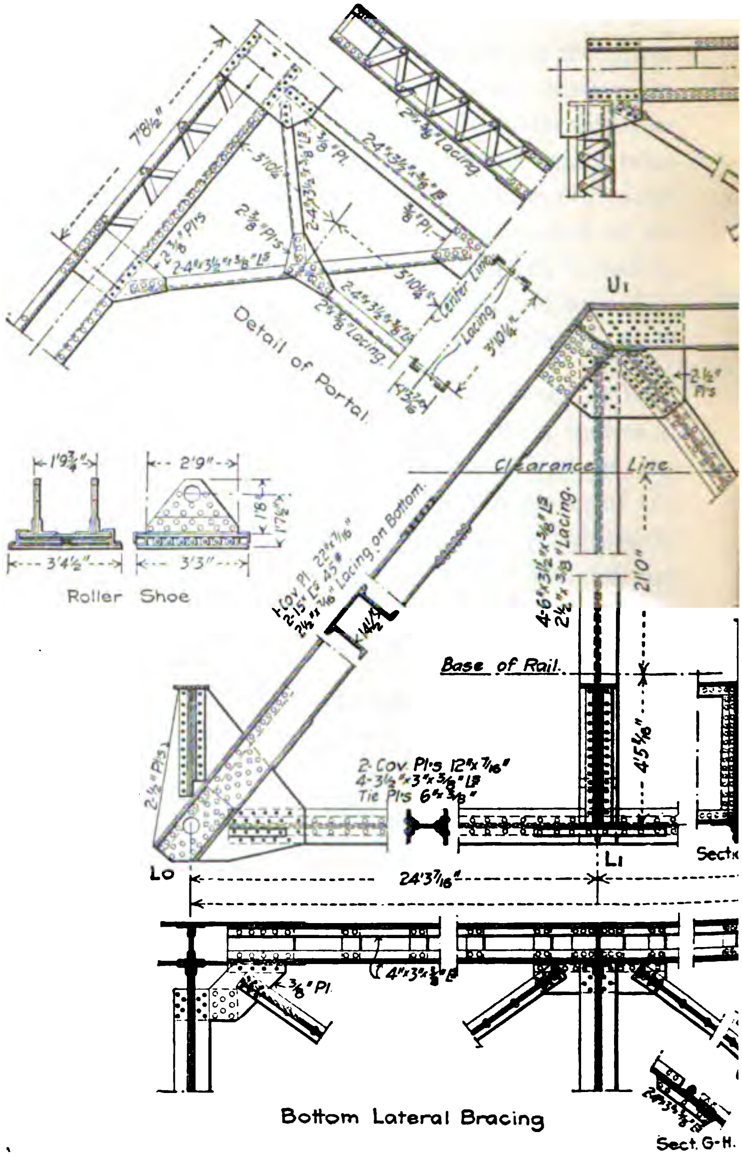
The general construction of the pedestals or shoes at both ends is indicated on the drawing. The rollers are 3 inches in diameter. In the first shoe the $3\frac{1}{2}$-inch vertical legs of the angles are planed to 3 inches. The web or pin plates of the pedestals as well as the bearing and bed plates are $\frac{3}{4}$ inch thick, while the connecting angles are $6'' \times 6'' \times \frac{5}{8}''$. The anchor bolts are of soft steel $1\frac{1}{4}$ inches in diameter and 2 feet long, having cold-pressed threads and foxed ends.

The ends of stringers and floor beams are faced as well as those of abutting compression members in order to secure perfect contact. All rivet holes are punched $\frac{1}{8}$ inch less and reamed to $\frac{1}{16}$ inch greater diameter than that of the rivet. All truss members are assembled in the shop and the field-rivet holes reamed to a perfect fit.

In the shorter spans the diagonals U_1L_2 are made of two channels laced together instead of two pairs of angles similarly connected, and the sections of the end post are balanced by riveting flats to the lower flanges of the channels.

A riveted truss bridge of the same type whose span is 150 feet is described and illustrated in Engineering News, vol. 40, page 114, Aug. 25, 1898. It was designed by the same engi-

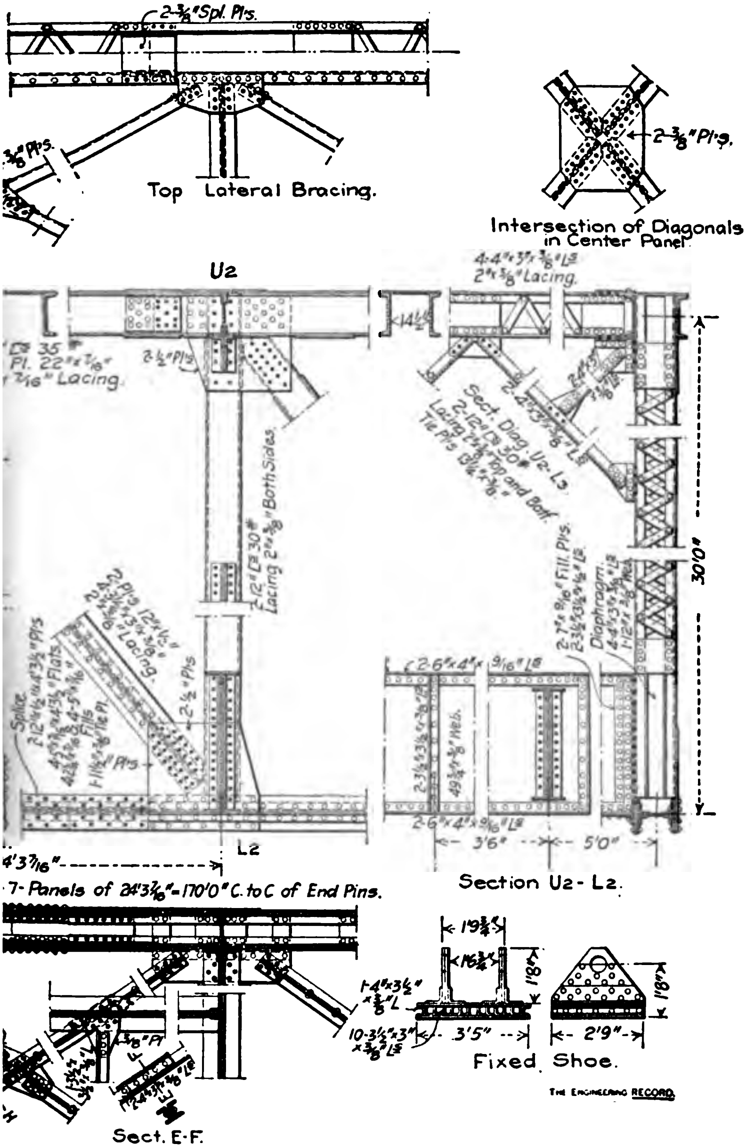
Single-track Through Riveted



PARTIAL DETAIL DRAWING OF ONE OF THE RIVETED

FOR THE VERA CRUZ AND

ıss Bridge. Span, 170 Feet.



RIDGES DESIGNED BY WADDELL AND HEDRICK IN 1899
ıCIFIC RAILWAY IN MEXICO.

neers for the Kansas City, Pittsburg and Gulf Railroad. Two of the principal differences in the details consist in the upper chord being laced on both the upper and lower sides, and in the portal bracing being a simple lattice girder, combined with knee braces, attached to the upper side of the end post.

ART. 127. DETAILS OF A BALTIMORE TRUSS.

Plate VII shows the details of a riveted truss bridge on the New York Central and Hudson River Railroad designed in 1910 under the direction of the Engineer of Structures. The plans were revised in 1911, and extra plans made for a heavier truss on one side to make provision for carrying two additional tracks in the future. The span is 156 feet 1¾ inches between centers of truss supports. The trusses have ten panels, a depth of 32½ feet, and are spaced 30 feet 8 inches apart. The stringers under each track are spaced 6½ feet center to center, and the nearer stringers under adjacent tracks have the same spacing. The bridge is designed for class E60 live loading.

The sub-verticals as well as the long suspenders consist of two pairs of angles connected by a web plate. The sub-diagonals have two channel shapes laced together, while both the main and counter diagonals are composed of built-up channels connected by lacing. The intermediate posts have the same composition as the long diagonals. The lower chords also consist of two built-up channels laced together, but with filler web plates added in two panels near the middle of the truss. The upper chords and end posts have the usual composition of two web plates, four flange angles, and a cover plate, with the addition of filler web plates throughout. In both cases, however, flats are riveted on the bottom of the lower flange angles to bring the neutral surface closer to the mid-depth of these members.

Fig. 182.—Bridge No. 451, over East Creek, Middle Division, New York Central and Hudson River Railroad. Span, 97 feet 2 inches.

Fig. 183.—Bridge No. 451, over East Creek, New York Central and Hudson River Railroad. Built in 1901.

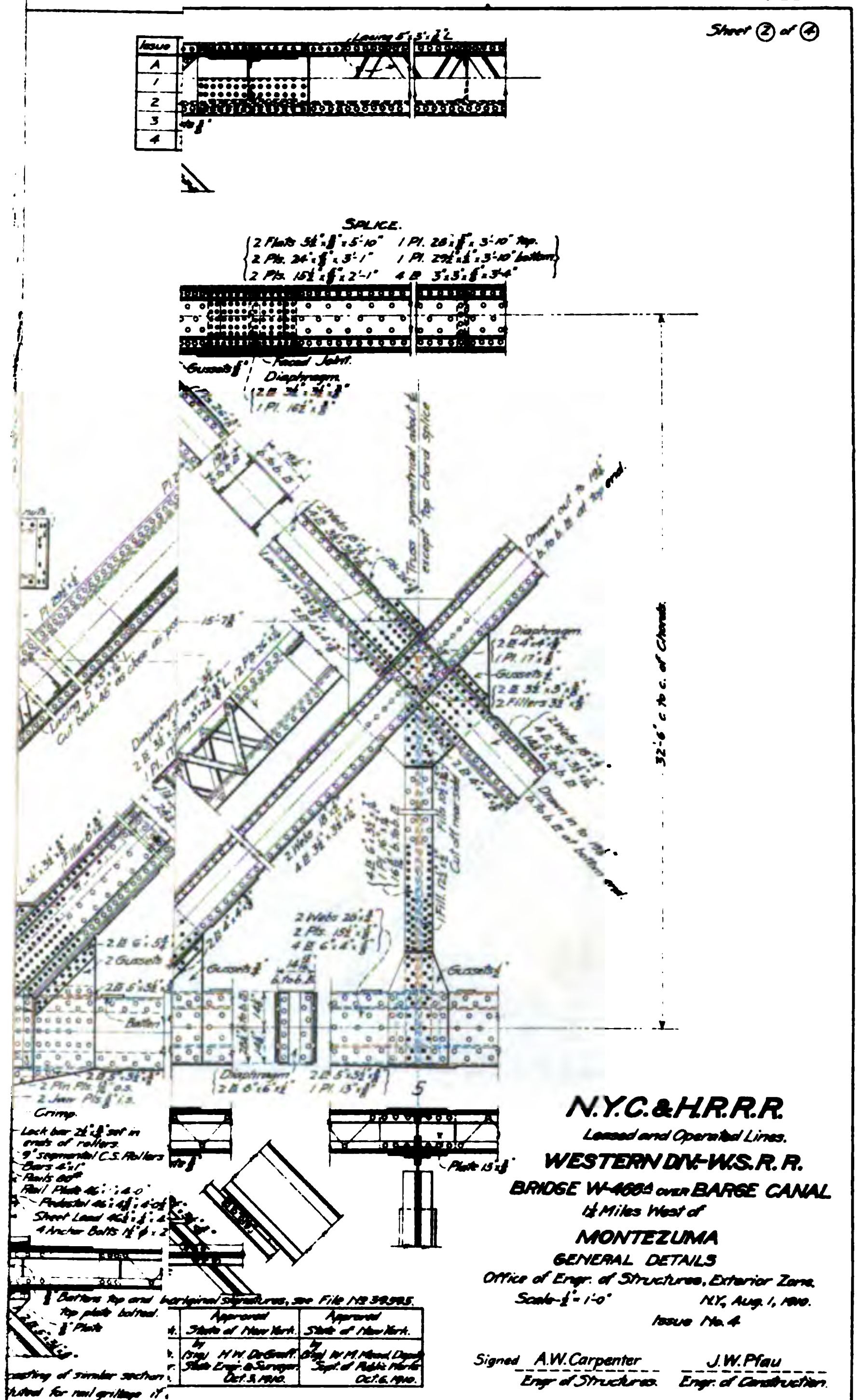
Angles with equal legs are used in the intermediate posts, long diagonals, upper chords, and end posts, while angles with unequal legs are employed in the sub-verticals, suspenders, and lower chords. The flanges are turned in on the sub-diagonals, vertical posts, and lower chords, while they are turned out on the long diagonals, upper chords, and end posts.

Intermediate transverse diaphragms are inserted in the upper chords and end posts, one being placed directly below the connection of the portal bracing. Since the intermediate posts do not extend into the upper chords, transverse diaphragms are placed also at these panel points. Short longitudinal diaphragms are inserted in the end posts opposite the lower portal strut, and the lower part of the portal brackets.

Main connecting or gusset plates one-half inch thick are used at the lower panel points of the truss except at the end, and at the joints midway between the chords. At the intermediate joints on the upper chord, their thickness is five-eighths of an inch, while those at the hip and end joints of the truss are three-quarters of an inch in thickness. Angle clips on the sides of a member are employed in many cases to shorten the riveted end connections of web members.

The widths of members, back to back of angles, measured transversely with respect to the truss, are as follows: suspenders, $16\frac{5}{16}$ inches; vertical posts, $15\frac{5}{16}$ inches; lower chords, $14\frac{1}{16}$ inches; long diagonals, $19\frac{1}{2}$ and $19\frac{1}{4}$ inches, the latter being in the counter-braced panels; upper chords, $19\frac{5}{8}$ inches; and end posts, $19\frac{5}{8}$ inches. The sub-diagonals are $16\frac{5}{16}$ inches wide, back to back of channels.

The floor beams are $39\frac{1}{4}$ inches deep, back to back of flange angles, and were made shallow since it was necessary to use a minimum floor depth on account of grades and the requirement


Sheet 2 of 4
Issue
A
1
2
3
4
SPLICE.
2 Flats 5½ x ⅝ x 5'-10" 1 Pl. 28 x ⅝ x 3'-10" top.
2 Pls. 24 x ⅝ x 3'-1" 1 Pl. 29½ x ½ x 3'-10" bottom
2 Pls. 15½ x ⅝ x 2'-1" 4 ⌊ 3 x 3 x ⅝ x 3'-4"
Faced Joint.
Diaphragm.
Truss Symmetrical about ℄ except Top Chord splice
32'-6" c. to c. of Chords.
Diaphragm.
Gussets
Crimp.
N.Y.C.&H.R.R.R.
Leased and Operated Lines.
WESTERN DIV.-W.S.R.R.
BRIDGE W-468A over BARGE CANAL
1½ Miles West of
MONTEZUMA
GENERAL DETAILS
Office of Engr. of Structures, Exterior Zone.
Scale-½"=1'-0"
N.Y., Aug. 1, 1910.
Issue No. 4
Signed A.W. Carpenter
Engr. of Structures.
J.W. Pfau
Engr. of Construction.
Approved State of New York.
Approved State of New York.
Oct. 3, 1910.
Oct. 6, 1910.

of raising tracks to provide a specified clearance over the canal. The distance from base of rail to the under clearance line is 3 feet 8 inches. The composition of the floor beams is a little unusual, but was designed to secure the maximum effective depth for stiffness. It is observed that in addition to a number of cover plates, flats are placed on the faces of the horizontal legs of the flange angles.

The vertical members of the lighter truss to which the floor beams are attached are given increased section areas to provide for the secondary stresses due to the deflection of the shallow floor beams. A part of the increased section is secured by extending the fillers under the floor beam connection angles, and by means of the extra plates above the gusset plates. Fillers are also extended on the main diagonals for the same purpose. A large increase was made in the section areas of the sub-verticals at panel point 1, in order to resist the secondary stresses due to the stiffness of the end post. Still further provision for secondary stresses due to the deflection of the floor beams is made by using slightly shorter lengths than the nominal lengths required for the upper lateral and transverse bracing. In erection the trusses are sprung inward one-half inch at the top to connect to the bracing, thus causing some bending moment in the verticals, which is relieved when the floor beams deflect under the live load traffic.

Lacing composed of angles is used on the upper chords, on the end posts, and on the long diagonals which are subject to bending due to the deflection of the floor beams. Angle lacing is used also in the upper lateral system. Lacing bars are used on the sub-diagonals, the posts, and the lower chord members. Only single lacing is employed throughout. Numerous other details are shown on Plate VII, which need no further description.

Fig. 184.—Delaware, Lackawanna, and Western Railroad Bridge over Susquehanna River, East of Binghamton, N. Y. Completed in 1906. Span 170 feet. Note the absence of lacing in many of the web members.

ART. 128. ANALYSIS OF WEIGHT

A profitable study of the details of a design may be made by preparing an analysis of the weight of the trusses and bracing, in a similar manner to that explained in Art. 101 for a pin truss bridge. The data in the following table were obtained from an analysis of 23 designs. The spans range from 98 feet 8 inches to 230 feet. Only three spans, however, exceed 170 feet, including one through Pratt truss span of 197.5 feet, and two Whipple truss spans of 230 feet. Of the series, 18 plans are those of single-track bridges, and 5 of double-track bridges; 13 are standard plans, and 10 are the plans of bridges for given locations; 2 have deck trusses, 3 have pony trusses, and 18 have through trusses. The equivalent uniform live load ranges from 4400 to 6625 pounds per linear foot of track, except in one case in which it is 3260 pounds. The dates of the standard plans or of the completion of the bridges are included in the first half of the decade following 1900.

The weights are divided into five parts: first, those of the main shapes which compose the members, including channels, angles, and plates, which extend nearly or quite the full lengths of the members, and with cross-sections designed to resist the respective stresses; second, those of the main connecting plates at the panel points of the trusses, and which perform the same function as the pins and pin plates in pin-connected trusses; third, those of the tie plates, lacing bars, and batten plates, which unite the main shapes of the members or hold them in line; fourth, those of other details, including plates and angles used at splices, in diaphragms, and at the connections of the lateral and transverse bracing, together with filler plates, the extra pin plates at the end panel points, etc; fifth, those of the rivet heads only, since no deduction is made for the rivet holes in plates and shapes which are subsequently filled by the shanks

of the rivets. The weights of all details in the end bearings are excluded in these analyses. All of the bridges had the ordinary type of open floor, except one which had a trough floor.

ANALYSIS OF WEIGHT OF TRUSSES AND BRACING.

The values given are percentages.

LINE.	DESCRIPTION OF BRIDGES.	MAIN SHAPES.	MAIN CONNECTING PLATES.	TIE PLATES AND LACING.	OTHER DETAILS.	RIVET HEADS.
1	14 Through Pratt and Warren Truss Bridges	73.9	7.1	8.0	7.0	4.0
2	3 Pony Pratt and Warren Truss Bridges	73.4	9.4	6.1	6.9	4.3
3	2 Deck Warren Truss Bridges . .	81.8	4.5	5.0	4.3	4.3
4	1 Through Baltimore Truss Bridge	77.2	7.9	6.1	4.6	4.2
5	Average for 20 Preceding Bridges	74.8	7.2	7.3	6.6	4.1
6	2 Through Whipple Truss Bridges	70.0	8.7	11.2	6.3	3.8
7	1 Through Quadruple Warren Truss Bridge	68.5	5.9	12.3	9.2	4.1
8	Average for 23 Preceding Bridges	74.1	7.3	7.9	6.7	4.1
9	18 Single-track Bridges	74.2	6.8	8.1	7.0	3.9
10	5 Double-track Bridges	73.8	8.9	7.0	5.7	4.6
11	9 Designed by Consulting Engineers	74.5	6.4	8.7	6.8	3.5
12	7 Designed by Railroad Bridge Departments	73.7	7.7	7.1	7.2	4.2
13	4 Designed by Engineers of Bridge Works	77.1	8.0	4.5	5.3	5.1
14	Maximum Values for Single Bridges	81.8	11.0	14.3	10.5	6.1
15	Minimum Values for Single Bridges	68.1	3.0	3.8	2.9	1.8

The table shows that there is but little difference between the results for through and pony trusses. The main connecting plates are relatively heavier since their thicknesses were the same as those of some of the heavier through trusses with

longer spans. The advantage of deck spans is evident on comparing the percentages in line 3 with those in lines 1, 2, and 4. Lines 6 and 7 indicate the effect of using multiple intersection trusses with more numerous web members of small section and with shorter panels. The through quadruple Warren truss span with 9 panels has a length and live loading intermediate between two 6-panel through Pratt truss spans. The weight of trusses and bracing for the lattice truss is 93 500 pounds, and for the two Pratt trusses is 70 000 and 80 600 pounds respectively. The weights of the entire spans, including single tracks of practically equal weight per foot, are 327 700, 269 700, and 295 100 pounds respectively. It must be remembered, however, that the shorter panels of the lattice trusses permit the use of a much shallower floor system than that in the Pratt truss spans. Lines 11, 12, and 13 indicate some effect of the personal equation in designers.

The highest percentages for main connecting plates occur in pony trusses, Whipple trusses, and in double-track bridges. In two cases where single- and double-track bridges have the same span and live loading, and are included in the same set of standards, the total weight of the connecting plates for the double-track bridges is about three times that for the single-track bridges. Web members composed of two pairs of angles require smaller widths of connecting plates than those consisting of laced channels. The lowest percentage occurs in a deck Warren truss in which there are no sub-vertical suspenders.

The highest percentage for tie plates and lacing occurs in a bridge in which these details are employed in every member of the trusses and upper laterals. The next in order are the multiple intersection trusses. The lowest percentage occurs in a deck Warren truss without suspenders.

The highest percentage for other details occurs in single-track bridges having heavy diaphragms in the verticals opposite the floor-beam connections. The use of filler plates between some members and the connecting plates also tends to increase the percentage for this group of details. The lowest percentage is in a pony truss which has no upper lateral system. Since the connections of the lateral and transverse bracing are included in this division of the weight, the character of such bracing as well as its entire absence will materially modify the percentage for these minor details. It should be added, however, that another pony truss stands third in the list of 23 bridges with a percentage of 9.9.

The highest percentage for the weight of rivet heads occurs in a Pratt truss span, 100 feet long, in which every vertical consists of a pair of angles united by a web plate, and the upper chord is built up of plates and angles, and with a cover plate. The lowest percentage occurs in a Pratt truss span 120 feet long in which all the members have tie plates and lacing, and the upper chord consists of two channels laced on both top and bottom.

It is also desirable to compare the final weights of different classes of main truss members, including their details, with their theoretic weights, which are obtained by considering only the main shapes composing the members, and assuming their lengths to be the distances between the corresponding panel points. The ratio of the final to the theoretic weight is found to vary as follows: For intermediate posts and suspenders, from 1.61 to 1.03; for diagonals and lower chords, from 1.45 to 1.03; for upper chords and end posts, from 1.49 to 1.07; and for all truss members, from 1.34 to 1.10. The corresponding average ratios are 1.25, 1.20, 1.21, and 1.21. These averages are found more uniform than those for pin-connected trusses.

An indication of the extreme differences in ratio for a given bridge may be observed from the following values for the classes of members arranged in order as given above: 1.61, 1.45, 1.09, and 1.34; 1.03, 1.23, 1.49, and 1.29. The following two examples show the least variation: 1.25, 1.21, 1.25, and 1.23; 1.27, 1.29, 1.25, and 1.27. The value of such ratios in designing trusses and details is explained in Art. 101.

In the 23 bridges, whose analyses for trusses and details are considered in this article, the entire weights of the spans, exclusive of end bearings, are distributed as follows: trusses and bracing, 68.2 to 40.7 percent; steel floor system, 41.7 to 0.0 percent; and track, 32.6 to 13.6 percent; the corresponding averages being 52.85, 24.74, and 22.41 percent. The largest percentage for the steel floor system is in a double-track bridge with a trough floor, while one of the deck spans had no floor system, the ties being supported directly by the upper chords of the trusses. If these two cases are excluded, the weight of the steel floor system, all of the ordinary open type, varies from 33.7 to 18.2 percent. The three remaining values below 22.0 percent occur in bridges with very short panels and multiple intersection trusses.

It is to be remembered that, if the same specifications and the same regard for ultimate economy in construction and maintenance had been used in the design of these 23 bridges, the variation in the results would be considerably reduced.

ART. 129. RIVETED TRUSS BRIDGES — REFERENCES.

The selected articles included in the following references to engineering periodicals contain descriptions and illustrations of the details of modern riveted railroad bridges and of some standard designs.

Bridge Work on the Kansas City, Pittsburg, & Gulf Railroad. Eng. News, v. 40, p. 114, Aug. 25, 1898.

Maiden Lane Bridge of the New York Central and Hudson River Railroad, Albany, N. Y. Eng. Rec., v. 40, p. 498, Oct. 28, 1899.

Standard Bridge Plans of the Northern Pacific Railway. By RALPH MODJESKI. Eng. News, v. 45, p. 60, Jan. 24, 1901; Eng. Rec., v. 43, p. 175, Feb. 23, 1901.

American Bridges in Mexico. Eng. Rec., v. 44, p. 196, Aug. 31, 1901.

Erection of the Northwest Miramichi Bridge, Newcastle, N. B.; Intercolonial Railway. By H. D. BUSH. Trans. Can. Soc. C. E., 1903, v. 17, p. 77.

Floor Beam and Lateral Street Connections to Riveted Trusses. Eng. Rec., v. 49, p. 416, Apr. 2, 1904.

One Hundred-foot Pony-truss Single-track Through Span with Shallow Floor. Eng. Rec., v. 49, p. 427, Apr. 2, 1904.

Fraser River Bridge, British Columbia. Eng. Rec., v. 49, p. 582, May 7, 1904.

Standard Bridges on the Harriman Lines. R. R. Gaz., v. 39, pp. 88, 130, 162, 188, and 200, July 28, Aug. 11, 18, 25, and Sept. 1, 1905.

Through-truss Short-span Double-track Bridge; New York, New Haven, & Hartford Railroad. Eng. Rec., v. 52, p. 464, Oct. 21, 1905.

End Panel Construction of a Skew Bridge; New York, Chicago, & St. Louis Railroad. Eng. Rec., v. 53, p. 481, Apr. 14, 1906.

New Belt Railway at Kansas City. Eng. News, v. 58, p. 518, Nov. 14, 1907.

McKees Rocks Bridge; Pittsburgh & Lake Erie Railroad. Eng. Rec., v. 58, p. 246, Aug. 29, 1908.

Replacing the Clyde River Bridge of the West Shore Railroad. Eng. Rec., v. 58, p. 463, Oct. 24, 1908.

New Bridge crossing the Mississippi at Clinton, Ia.; Chicago & Northwestern Railway. By F. H. BAINBRIDGE. Eng. News, v. 61, p. 63, Jan. 21, 1909.

Central of New Jersey Bridge Renewal at Bethlehem. Ry. Age Gaz., v. 46, p. 251, Feb. 5, 1909.

Long-span Riveted-truss French River Bridge; Canadian Pacific Railway. Eng. Rec., v. 60, p. 245, Aug. 28, 1909; Eng. News, v. 60, p. 85, July 23, 1908; Trans. Can. Soc. C. E., 1908, v. 22, p. 204.

Reconstruction of the Steubenville Bridge; Pittsburgh, Cincinnati, Chicago, & St. Louis Railway. Eng. Rec., v. 62, p. 596, Nov. 26, 1910.

Construction and Reconstruction of the Coteau Bridge; Grand Trunk Railway. Eng. Rec., v. 62, p. 628, Dec. 3, 1910.

Renewal of the Kentucky River High Bridge, Cincinnati, New Orleans, & Texas Pacific Railroad. By H. H. STARR. Eng. News, v. 65, p. 518, Apr. 27, 1911; Eng. Rec., v. 62, p. 774, Dec. 31, 1910.

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